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too numerous to mention separately, took the time and effort away from their normal duties to gather and supply the data required.

APPENDIX.—REFERENCES

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PENETRATION RESISTANCE FOR DRIVEN PILING

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INTRODUCTION

It is one task to specify length of piling that will satisfy normal design requirements for adequate load bearing capacity and limited settlement. It is quite another to decide whether the piles will actually be driven to that depth, or beyond, and to determine how much time and effort will be expended in the process. Design requirements can be satisfied despite recognized shortcomings in analytical methods because of the margin for error permitted by choice of a suitable factor of safety. We compute ultimate capacity, but never approach it to test our predictions. With penetration resistance, however, we must predict a result we will encounter. If a reliable method of predicting penetration resistance were developed, driven length and driving time could be estimated, and surprises during construction, along with their associated costs, could be minimized.

ANALYTICAL APPROACH TO PENETRATION RESISTANCE PREDICTIONS

Methods of predicting pile penetration resistance may be analytical or empirical. In the latter case a large body of experience is summarized in some form, but without a theoretical framework that will allow its extrapolation to conditions other than those for which the experience was gained. Such results have limited application. A purely theoretical approach is also deficient in that it has not been tested against experience. The most logical method of predicting penetration resistance would likely be a combined empirical-analytical approach that incorporates field experience, but interprets it in light of a theoretical framework which considers the mechanics of pile driving.

Two methods of analysis having the same general framework were investigated in this study. Basic to both methods is the assumption that for a given pile-soil system, the bearing capacity of the pile at a given depth is the same regardless

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of the hammer used to drive the pile to that depth. The first method is based on analysis for static bearing capacity, while the second is based on analysis for dynamic bearing capacity, or driving resistance.

Method Based on Static Soil Bearing Capacity.—The first method proposed is outlined as follows: (1) Compute the bearing capacity, using conventional static analysis, of the pile to be driven, for several depths within the subsurface profile; (2) assume equality of the computed static capacity and dynamic capacity; and (3) compute anticipated penetration resistance at the several depths, using a dynamic pile driving formula and the assumed dynamic capacity. The first step requires knowledge of the engineering properties of the soils to be penetrated. Even with representative properties, bearing capacity calculations produce better results for some types of soils than others. In clays, for instance, it has been found that estimates of static capacity are quite reliable. In sandy soils or in layered soils, the static analysis procedure is less reliable. Accordingly, there is a weakness in the first step of the proposed method which must be overcome if it is to offer a reliable approach.

Regarding the second step, it is known that the assumption of equality of static and dynamic capacities is questionable. In clay soils and loose fine sands significant excess pore pressures are developed during driving. After termination of driving the pressures dissipate, resulting in pile setup, as evidenced by increased penetration resistance during re-driving. Obviously, capacity during and following driving are not the same.

The validity of the third step is dependent on the particular dynamic formula used to relate penetration resistance to dynamic capacity. Conventional pile driving formulas, which rely on rigid body mechanics to make dynamic calculations, are commonly used. It has been shown, however, that more sophisticated dynamic analysis based on the use of the wave equation will give more reliable predictions of capacity in most soil types (6,10). Some of the conventional formulas are reasonably reliable, particularly for piles driven in sands. Among these is the Hiley formula, which includes certain corrections for pile, soil, and hammer characteristics. Such corrections are not included in the more commonly used Engineering News formula. Wave equation analysis permits inclusion of the broadest representation of pile, hammer, and soil characteristics.

Method Based on Penetration Resistance Records.—The aforementioned method is based on soil properties alone. If pile driving records were available from the site under study, or from a site where soil conditions were similar, use may be made of these records to assist with penetration resistance predictions. The availability of pile driving records precludes the need for assuming that calculated static bearing capacity is equal to dynamic capacity; the driving record represents dynamic capacity and may be used to advantage. The method outlined previously could therefore be modified as follows:

1. Compute the dynamic bearing capacity (driving resistance) for several depths using a dynamic pile driving formula and representative available historic penetration resistance records.
2. Assume that the computed driving resistance of the piles to be driven will be equal to that of piles for which records are available, for equal penetration depths.
3. Compute a penetration resistance curve for the new piles from the dynamic

capacities calculated from historical records. Make the calculations for driving equipment proposed for use on the new project. The same dynamic formula used in step 1 should be used here.

This approach improves the procedure based on static bearing capacity analysis in two ways. First, it permits predictions to be made without the need for computing the actual bearing capacity of the piling. A bearing capacity (driving resistance) is computed which is derived from a dynamic analysis only. It need not be the actual capacity. Second, the need for assuming equality of static and dynamic capacities is avoided. An estimate of bearing capacity, usually based on static analysis, may be required to perform the first and third steps, depending on which dynamic formula is used. The energy losses included in the Hiley formula, for example, are calculated using an assumed driving resistance. Wave equation analysis requires an estimate of the percentage resistance offered by the pile tip and the distribution of side resistance.

The principal shortcoming of this method, of course, is that driving records for the same or similar type piling from a site having the same or very similar soil conditions must be available. To fully develop this method would require that a large enough body of historical driving information be assembled so that representative values for all conventional hammer types, soil conditions, and pile types are available. Attempts to do this have been made before (8).

INVESTIGATIONS WITH FIELD DATA

In order to examine the validity of the methods of penetration resistance prediction proposed, records were gathered from five projects on the West Coast. In each case a considerable volume of information had been obtained during construction. Information sources were limited by the need for sufficient information to perform wave equation analysis and conventional dynamic analysis. Adequately defined subsurface soil conditions were required to permit conventional static bearing capacity calculations. Most important, driving records for

TABLE 1.—Projects Selected for Analysis^a

Project designation (1)	Pile hammers (2)	Pile types (number) (3)	Soil profile (4)
A—Portland, Oreg.	Vulcan 50C Vulcan 65C	10-in. Steel Pipe (25) 10-in. Steel Pipe (167)	Silt
B—Portland, Oreg.	Vulcan 50C Vulcan 65C	Timber (200) 10-in. Steel Pipe (23)	Sand, Silt, Clayey Silt
C—Seattle, Wash.	Vulcan 104C Vulcan 014	16-1/2-in. Concrete (28) 16-1/2-in. Concrete (27)	Silt, Sand
D—Richmond, Calif.	Delmag D30 Delmag D36	14-in. Concrete (5) 14-in. Concrete (6)	Clay, Sand
E—South San Francisco, Calif.	Kobe K42 Delmag D44	12-in./14-in. Concrete (24) 12-in./14-in. Concrete (107)	Silt, Clay, Sand

^aProject identification by owner and specific location has been omitted by request.
Note: 1 in. = 25.4 mm.

two different hammers used on the same project site, each on a representative number of piles, were necessary. This would allow examination of the proposed predictive methods by making predictions for one hammer based on data obtained for the other hammer, and then comparing prediction with field observation.

Project Studies.—The projects selected are described briefly in Table 1. Soil profiles and engineering properties used in the analyses are included in Figs.

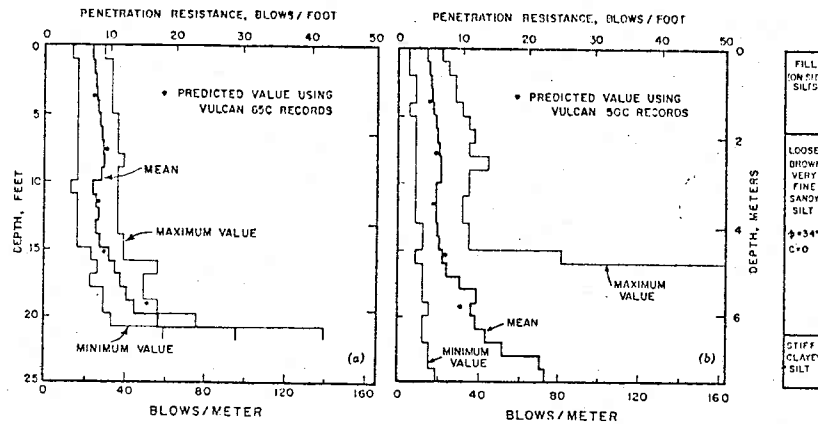


FIG. 1.—Penetration Resistance Versus Depth for Project A for: (a) Vulcan 50C; (b) Vulcan 65C (Predictions Made Using Wave Equation Analysis)

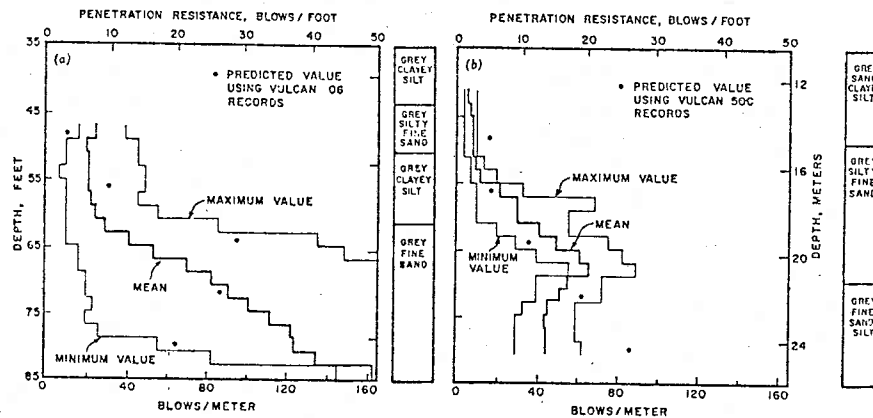


FIG. 2.—Penetration Resistance Versus Depth for Project B for: (a) Vulcan 50C; (b) Vulcan 06 (Predictions Made Using Wave Equation Analysis)

1-6 of the next section. As indicated by Table 1, an attempt was made to include a range of hammer and piling types and soil conditions. A small number of projects representing a broad range of conditions seemed the most appropriate way to approach an investigation of the feasibility of predictive methods. Brief descriptions of each project site follow; more complete data, including models

for wave equation analysis, are available in the original project report (11) and a thesis by the first writer (7).

Project A.—Ten-inch (250-mm) nominal diameter, closed-end steel pipe piling were driven 15 ft-30 ft (4.5 m-9.1 m) through silt fill and in-situ sandy silt

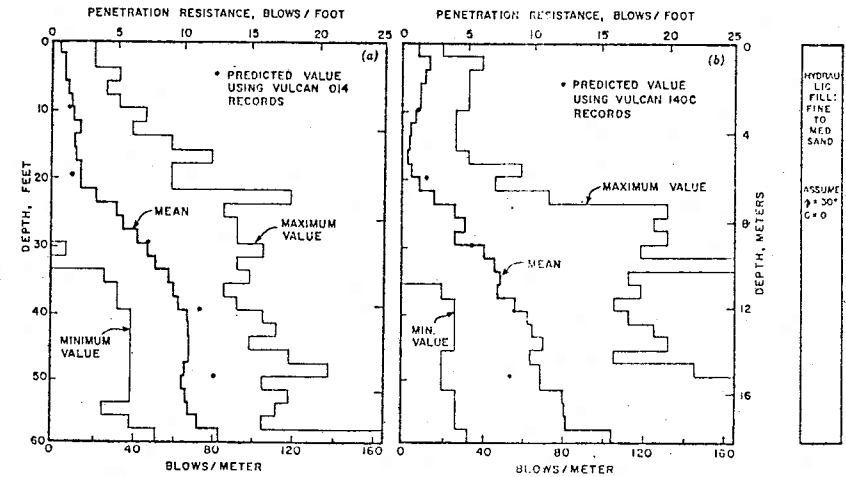


FIG. 3.—Penetration Resistance Versus Depth for Project C for: (a) Vulcan 140C; (b) Vulcan 014 (Predictions Made Using Wave Equation Analysis)

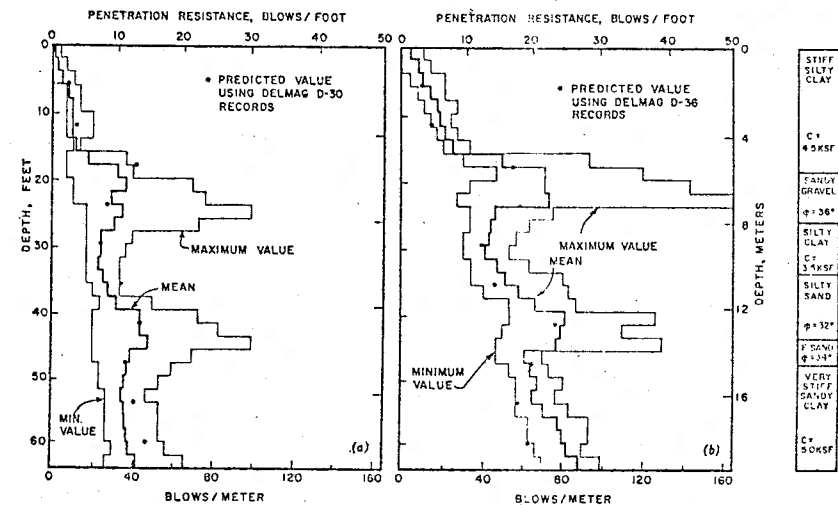


FIG. 4.—Penetration Resistance Versus Depth for Project D for: (a) DELMAG D-36; (b) DELMAG D-30 (Predictions Made Using Wave Equation Analysis)

to end bearing in decomposed bedrock. The bedrock surface at the site slopes from one end of the project to the other. Individual driving records were therefore selected from a specified area with average depth to bedrock of 20 ft (6.1

m). This study was not concerned with end bearing, comparisons were made only for penetration resistance above bedrock.

Project B.—The two sites of this project, separated by a distance of 1,110 ft (340 m), presented the most diverse soil-pile-hammer system of the projects

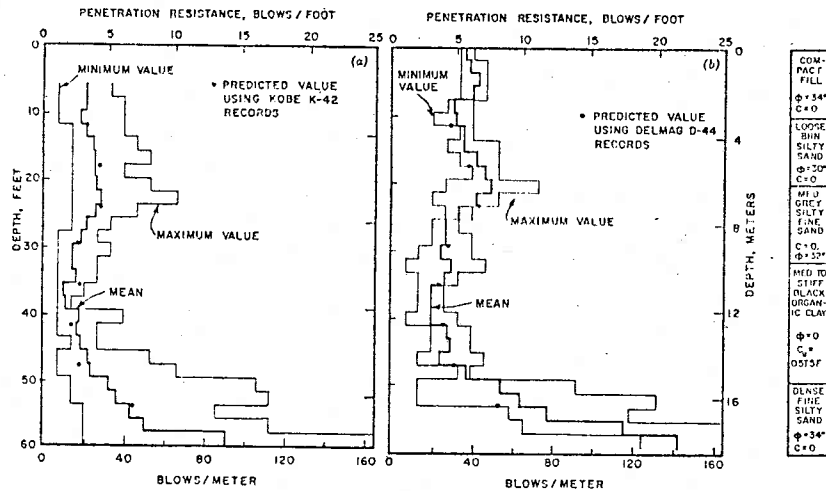


FIG. 5.—Penetration Resistance Versus Depth for Project E, 12-in. piling, for: (a) DELMAG D-44; (b) KOBE K42 (Predictions Made Using Wave Equation Analysis)

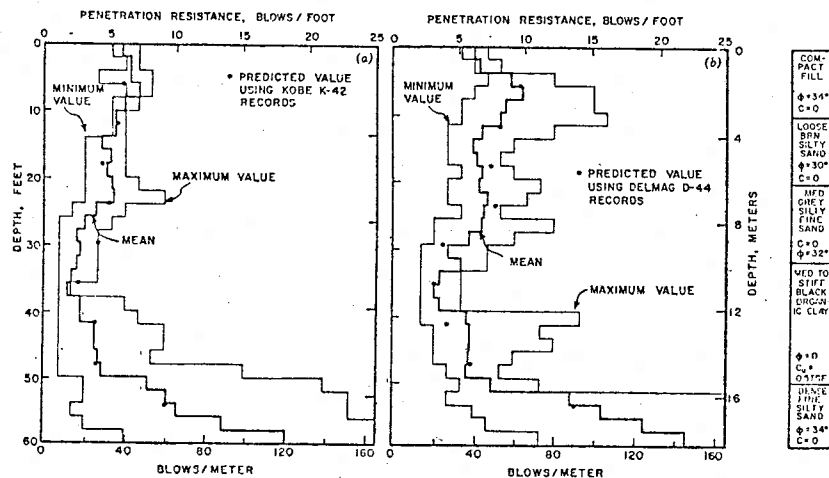


FIG. 6.—Penetration Resistance Versus Depth for Project E, 14-in. piling, for: (a) DELMAG D-44; (b) KOBE K42 (Predictions Made Using Wave Equation Analysis)

selected. The piling at both sites were driven through alluvial deposits of fine sands, silts, and clayey silts to friction bearing in sands and silty sands. At one site treated timber piles were driven using a differential acting steam hammer, while at the other site steel pipe piles were driven using a single acting steam

hammer. Blow counts were not recorded for the upper strata, but were recorded continuously after a depth of 40 ft–45 ft (12 m–14 m) was reached. Though the soil profiles are dissimilar, partial analysis was made to investigate the importance of having similar subsurface conditions and pile types in the proposed methods of analysis.

Project C.—The 16.5-in. (420-mm) octagonal prestressed concrete piles were driven on the outboard side of a dike constructed of medium sand. The piling at each of the two bents analyzed varied from 65 ft–120 ft (20 m–37 m) in length, but only piles from 102 ft–120 ft (31 m–37 m) in length were included in the analysis. Different hammers were used for each of the bents. Driving logs from the two bents, which are separated by a distance of 100 ft (30 m), were compared on the basis of depth of penetration rather than elevation along the dike slope.

Project D.—The 12-in. (305-mm) square, approx 66-ft (20-m) long concrete piles were driven through alternating strata of clay and sand. Only 11 driving records were available for analysis, five for one hammer and six for the other. The piling were driven at an excavated site, thus the excavated depth of 20 ft (6.1 m) corresponded to zero depth on the driving records.

Project E.—Diesel hammers were used to drive 12-in. and 14-in. (305-mm and 360-mm) square prestressed concrete piling through layers of fill, loose silty sand, and stiff organic clay to bearing in dense silty sand. Depth of penetration varied from 58 ft–84 ft (18 m–26 m). For analysis Project E was treated as two separate projects, one including 12-in. (305-mm) piling and the other 14-in. (360-mm) piling. Though the possibilities of analysis are increased by the inclusion of a second pile size, the available data precluded making such analyses. The Kobe K42 hammer was used to drive test piling only, a total of eight piles, including four of each size.

Analysis Procedure.—The procedure followed for analysis of project records was to summarize the driving records for each of the hammers used on each of the projects. The driving records for each foot of driving were averaged to develop a mean or typical pile driving curve for each hammer. The mean length and mean driven length for each pile type was computed. A summary or model profile was developed using boring logs and laboratory test data.

In Figs. 1–6 are shown the summary profile and the mean, maximum, and minimum values of penetration resistance observed for the driving records analyzed. These summary records were used for predictions of the performance of each hammer. These predictions were based on the observations of the performance of the other hammer and static bearing capacity analysis using the methods outlined previously. References to predicted values shown in Figs. 1–6 are for the particular case of analysis based on driving records and the wave equation. The filled circles are values predicted from the mean driving record of the indicated hammer. Predicted values using driving records for one hammer are plotted for comparison with the observed mean driving record of the other. In Fig. 1, for example, the driving record for the Vulcan 50C is shown along with predicted points for that record made using the driving record of the Vulcan 65C, and vice-versa. A summary evaluation of these predictions and those made by other methods is presented in following sections.

Dynamic Formulas for Analysis.—Either proposed method requires conversion of capacity (either static or driving resistance) versus depth curves to penetration

resistance versus depth curves. In this study conversions were made using the Engineering News and Hiley formulas and wave equation analysis. The two dynamic formulas used are well known to foundation engineers; therefore their formulation is not presented herein. The reader is referred to Ref. 3. All wave equation analyses were made using a computer program developed at Texas A & M University; the reader is referred to T. C. Edwards' utilization manual (4) for a complete description of the program.

For the dynamic formulas the manufacturer's rated energy as modified by an efficiency factor was used. Where diesel hammers were employed, adjustments were not made to account for the explosive force, but the efficiency factors listed by Chellis (3) were used. The temporary compression allowance for pile head and cap and pile included in the Hiley formula were estimated using static capacity estimates. These same estimates provided a means for estimating the percentage point resistance and side resistance required for wave equation analysis. The temporary compression allowance for the soil, or quake, was assumed to be 0.1 in. (2.54 mm), independent in soil type or difficulty of driving. Chellis' values of coefficient of restitution were assumed (3).

For wave equation analysis hammer and pile properties corresponded to those used in the conventional dynamic formulas, with the exception of the diesel hammers' explosive force. In the wave equation computer program the work done on a pile by an explosive force is accounted for by an explosive pressure. Manufacturers' values of explosive force were assumed for the analyses. Stiffnesses of capblocks, cushions, and pile sections were calculated using their material properties. If capblock and cushion material were not adequately specified in project reports, parameter studies were made. Generally, it was found that the analysis was insensitive to large variations of stiffness (two orders of magnitude) for the low resistances encountered during driving. Coefficients of restitution and elastic moduli were taken from tabulated values (2, 5, 9) unless the information was available in project reports.

Soil parameters required for wave equation analysis are not well known. The soil quake was assumed constant for all soil profiles. The same value was used for the soil at both the side and tip of the pile. The value of 0.1 in. (2.54 mm) corresponds to that used in Hiley formula analysis. The value of the damping coefficient was assumed to be 0.1 sec/ft and 0.3 sec/ft (0.3 s/m and 1.0 s/m) for sand and clay, respectively, encountered at the tip of the pile. For soil at the side of the pile the damping constant was assumed to be one-third of that at the point. Where the pile was driven through both sands and clays, a value of the damping constant was computed based on the sand and clay values and the percentage embedment in a given soil type. This procedure, as well as the sand and clay values stated, is based on the work of Lowery, Edwards, and Hirsch (10), and the recommendations of Smith (12).

The relative percentage of side and tip soil resistance was calculated using static capacity estimates. In some instances, particularly where silts were penetrated, separate analyses were made using drained and undrained shear strength parameters. The wave equation analysis was then made using tip resistance estimates based on both analyses. For these cases comparison of the driving resistance versus set curves revealed no significant differences. Bowles (2) and Forehand and Reese (6) have shown the sensitivity of wave equation analysis to the percentage tip resistance. We did not conduct parameter studies

to evaluate our assumptions concerning distribution of load in the soil supporting the piles. We relied on static estimates by generally accepted methods to provide useable data.

RESULTS OF ANALYSIS

Predictions of penetration resistance for Projects A, B, C, D, and E were obtained using the two methods proposed. Comparisons of predicted and observed penetration resistance values were summarized using two statistical methods for all projects except B. (It was eliminated because of obvious unsuitable predictions resulting from soil conditions that were not comparable.) First, predicted and observed values for the same depth of penetration were plotted on a scatter diagram. A straight line relationship was fit to the data using linear regression analysis (least-squares method). The relative scatter of data points about this best fit relationship is expressed by the correlation coefficient, r . Second, the ratio of predicted to mean field penetration resistance was calculated for each data point. Mean \bar{x} , coefficient of variation v , and range between maximum and minimum ratios for a set of data points were determined. The coefficient of variation is the ratio of the standard deviation, s , and the mean; it is a measure of the dispersion of a set of ratios about their mean, relative to that mean. The necessary relationships for calculating these statistics are found in any standard text on statistical methods (1), and are not presented herein.

Predictions Using Static Soil Bearing Capacity.—Static capacity predictions were made for the piling on the projects and these static capacity curves were converted to predicted penetration resistance curves using wave equation analysis, the Hiley formula and the Engineering News formula. The results of these analyses are shown in Figs. 7(a)–7(c), respectively. On these figures the data points are plotted by project along with the least-squares relationship for all projects and a given dynamic formula. A perfect fit would be represented by the solid line being superimposed on the dashed line shown; in other words, the mean predicted penetration resistance would equal the mean observed penetration resistance. The scatter of data points about the best fit lines is large, and the slopes of the best fit lines indicate predictions are lower than observed values.

These qualitative observations are mirrored by the statistics presented in Table 2. The slopes of the regression lines are 0.86, 0.73, and 0.26 for wave equation analysis and the Hiley and Engineering News formulas, respectively. Similarly, the mean ratios, \bar{x} , of predicted to observed penetration resistance are 0.96, 0.83, and 0.41. Ideal values for both these parameters would, of course, be one. Though the mean ratio is nearly one for the wave equation approach, indicating its general reliability, the coefficient of variation, v , of 0.48 indicates a given prediction is not necessarily reliable. From the tabulated statistics, however, one would conclude that the use of wave equation analysis in the predictive method results in more reliable predictions than the use of either conventional dynamic formula. Also, the Hiley formula is apparently a better predictor than the Engineering News formula. This hierarchy of reliability would be expected because of the relative degrees of rationality of the three formulas. All three approaches based on static capacity calculated from soil data, however,

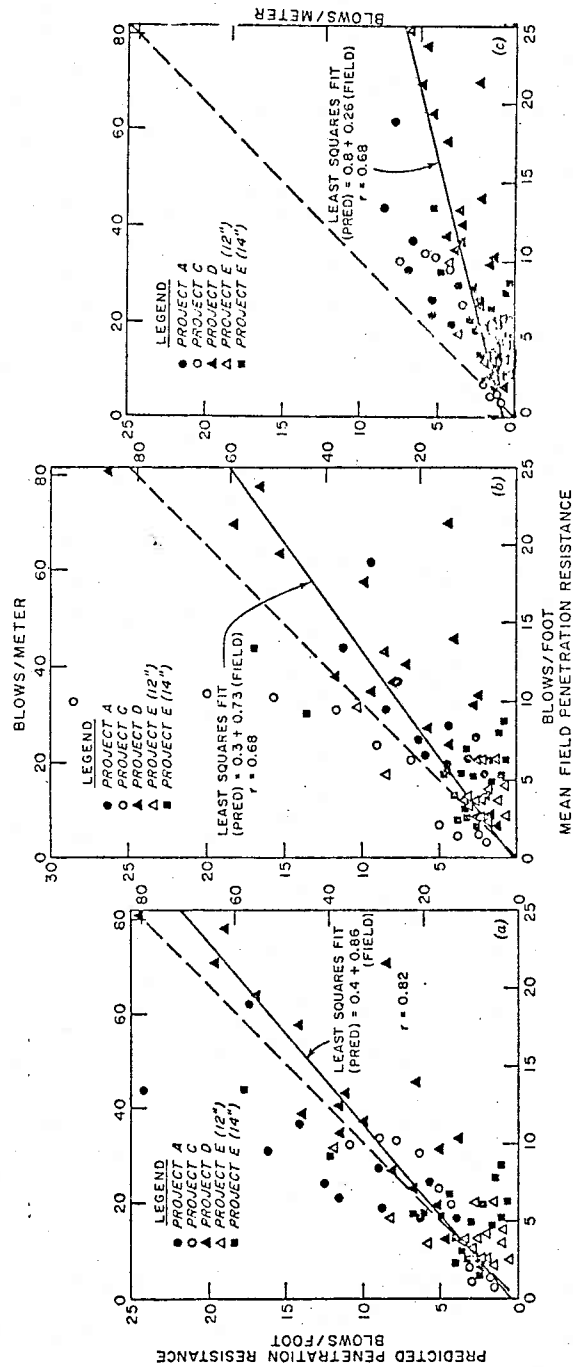


FIG. 7.—Predicted Versus Mean Field Penetration Resistance for All Projects, Using: (a) Wave Equation Analysis and Soil Bearing Capacity; (b) Hiley Formula and Soil Bearing Capacity; (c) Engineering News Formula and Soil Bearing Capacity

are not particularly reliable. The discrepancies are probably due, for the most part, to the difference between static capacity and driving resistance (dynamic capacity) mentioned earlier.

Table 3 lists the mean ratio and coefficient of variation for each project by dynamic formula. As outlined in Table 1 previously, Projects A and C were in comparatively uniform strata of sands and silts, whereas Projects D and

TABLE 2.—Summary of Statistics for Predictions Using Soil Bearing Capacity

Dynamic formula (1)	Least-Squares Fit			Ratio of Predicted to Observed		
	Slope (2)	Intercept (3)	r (4)	n (5)	\bar{x} (6)	v (7)
Wave Equation	0.86	0.4	0.82	75	0.96	0.48
Hiley Formula	0.73	0.3	0.68	75	0.83	0.70
Engineering News Formula	0.26	0.8	0.68	75	0.41	0.56

TABLE 3.—Statistics by Project for Ratio of Predicted to Mean Field Penetration Resistance Using Soil Bearing Capacity

Project (1)	n (2)	\bar{x} (3)	v (4)	Range (5)
(a) Wave Equation				
A	11	1.29	0.31	0.77-1.81
C	10	1.20	0.48	0.71-2.38
D	20	0.89	0.29	0.39-1.35
E (12 in.)	16	0.80	0.54	0.29-1.58
E (14 in.)	18	0.85	0.59	0.14-1.75
(b) Hiley Formula				
A	11	0.64	0.31	0.35-0.88
C	10	1.89	0.35	1.11-3.00
D	20	0.61	0.39	0.21-1.06
E (12 in.)	16	0.68	0.56	0.21-1.56
E (14 in.)	18	0.73	0.68	0.09-1.63
(c) Engineering News Formula				
A	11	0.55	0.31	0.30-0.79
C	10	0.70	0.39	0.44-1.23
D	20	0.27	0.30	0.11-0.38
E (12 in.)	16	0.38	0.47	0.10-0.70
E (14 in.)	18	0.35	0.66	0.06-0.83

E were in layered systems using diesel hammers. It might be expected that more reliable predictions could be made for Projects A and C as more reliable static capacity estimates could be made. The limited number of analyses do not appear to support this contention. For wave equation analysis the mean ratio of predicted to observed penetration resistance is 1.20 and 1.29 for Projects A and C. It ranges from 0.80-0.89 for Projects D and E. Similarly, for the

conventional formulas the mean ratio is somewhat larger for Projects A and C than for Projects D and E, though both formulas seriously underpredict the observed penetration resistance. An explanation for these differences is not readily available; the number of variables is large. Inadequate static and dynamic capacity estimates and the inequality of static and dynamic capacities likely play a major role. Analysis of more projects would allow better isolation of

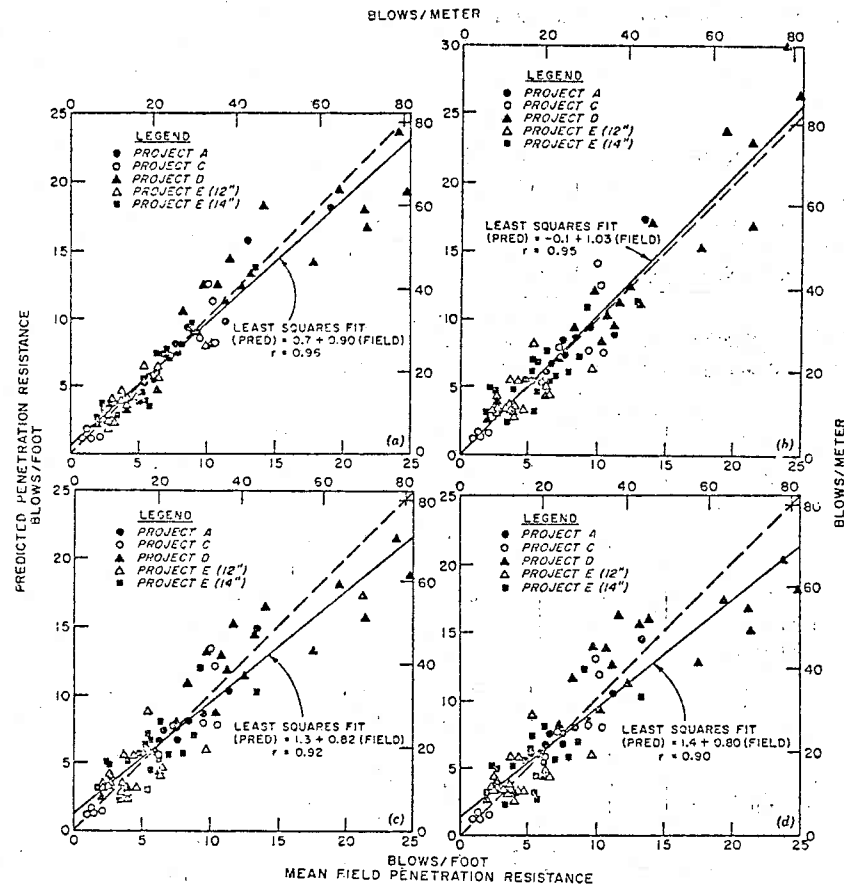


FIG. 8.—Predicted Versus Mean Field Penetration Resistance for All Projects, Using: (a) Wave Equation Analysis and Known Driving Records; (b) Hiley Formula and Known Driving Records; (c) Engineering News Formula and Known Driving Records; (d) Rated Energy Ratio and Known Driving Records.

the various elements of a hammer-soil system and investigation of their effects on the predictive method.

Predictions Using Penetration Resistance Records.—The projects selected for analysis were also evaluated using the second procedure proposed. Dynamic capacity (driving resistance) of the "typical" piling was calculated from the mean record for one hammer with a dynamic formula. It was assumed to be

representative for piling driven with the second hammer. The penetration resistance for the second hammer was then computed using the same formula. Wave equation analysis, the Hiley and Engineering News formulas, and a scaling method described subsequently were all employed.

The scaling method assumes that the hammer's energy output is the predominate parameter of the pile driving process. Evidence of this can be seen in viewing Figs. 1-6 presented previously. Particularly in Fig. 2, the driving records for hammers used in Project B, a relationship between a predicted record and the record employed to make that prediction can be seen. The two curves have essentially the same shape but the predicted curve is shifted to the right or left of the corresponding observed curve. The cause for the shift is the difference in energies of the two hammers: if hammer A has half the energy of hammer B, hammer A would drive a pile at about twice the penetration resistance that hammer B would drive it. This neglects, of course, other factors that enter into the mechanics of pile driving. But using this simple concept and knowing the penetration resistance for a pile driven by one hammer and the energy of a second hammer, the penetration resistance for the same pile driven in the same soil profile by the second hammer could be predicted.

The results of an analysis of the case histories employing this simple method are shown in Fig. 8(d). Results employing wave equation analysis, the Hiley and Engineering News formulas are shown in Figs. 8(a), 8(b), and 8(c) respectively. It is obvious that there is considerable improvement in our ability to predict penetration resistance if records are available. Slopes of the linear regression lines vary from 0.80 for the rated energy ratio approach to 1.03 for the Hiley formula approach. Mean ratios vary from 1.01 for wave equation analysis to 1.05 for both the Hiley and Engineering News formulas.

The scatter of data points about the regression lines for predictions based on driving records is contained within a much narrower band than for the predictions employing an estimated soil bearing capacity. Correlation coefficients listed in Table 4 vary from 0.90 for the rated energy ratio approach to 0.96 for the wave equation approach. Similarly, coefficients of variation range from 0.31-0.18 for the same approaches, respectively. Reliable predictions, then, can be made using any of the four approaches, though wave equation analysis and the Hiley formula result in better estimates of penetration resistance more often than do the less rational Engineering News formula and rated energy ratio formulation. This would be expected. The choice of whether to use the Hiley formula or wave equation approach, it appears, can be made according to the preference of the analyst.

A review of the statistics presented in Table 5 indicates that better predictions were made for Project A than for the other projects. The range of the predicted to observed ratio is 0.79-1.29 for Project A, inclusive of all approaches, while the range for the other projects is much higher, varying from 0.67-1.46 for Project C to 0.46-2.13 for Project E. As mentioned previously, this might be expected because of the differences in soil profiles and driving equipment associated with these projects. Compiling one representative profile for a layered system, particularly where strata are sloping or of nonuniform thickness, necessarily introduces approximations. This is compounded by the loss of efficiency experienced by a diesel hammer in driving piling in soft material, as compared to a steam hammer. The differences may also be statistical, since

of our driving records for one hammer were available to make the analyses for Project E, and only a total of 11 were available for Project D.

TABLE 4.—Summary of Statistics for Predictions Using Known Driving Record

Dynamic formula (1)	Least-Squares Fit			Ratio of Predicted to Observed		
	Slope (2)	Intercept (3)	r (4)	n (5)	\bar{x} (6)	v (7)
Wave Equation	0.90	0.7	0.96	75	1.01	0.18
Hiley Formula	1.03	-0.1	0.95	74	1.05	0.27
Engineering News Formula	0.82	1.3	0.92	74	1.05	0.30
Rated Energy Ratio	0.80	1.4	0.90	74	1.04	0.31

TABLE 5.—Statistics by Project for Ratio of Predicted to Mean Field Penetration Resistance Using Known Driving Records

Project (1)	n (2)	\bar{x} (3)	v (4)	Range (5)
(a) Wave Equation				
A	11	1.01	0.09	0.87-1.16
C	10	1.02	0.24	0.67-1.46
D	20	1.01	0.19	0.77-1.29
E (12 in.)	16	1.00	0.15	0.74-1.26
E (14 in.)	18	1.02	0.22	0.64-1.56
(b) Hiley Formula				
A	10	1.01	0.13	0.79-1.29
C	10	1.03	0.23	0.72-1.41
D	20	1.02	0.20	0.71-1.39
E (12 in.)	16	1.05	0.30	0.65-1.50
E (14 in.)	18	1.11	0.34	0.68-2.04
(c) Engineering News Formula				
A	10	1.00	0.09	0.88-1.13
C	10	1.03	0.22	0.75-1.35
D	20	1.02	0.24	0.69-1.43
E (12 in.)	16	1.06	0.35	0.61-1.65
E (14 in.)	18	1.13	0.37	0.56-2.13
(d) Rated Energy Ratio				
A	10	1.00	0.09	0.87-1.13
C	10	1.02	0.22	0.76-1.31
D	20	1.03	0.27	0.69-1.43
E (12 in.)	16	1.06	0.35	0.60-1.65
E (14 in.)	18	1.08	0.42	0.46-2.13

A further indication of the reliability of the wave equation approach is shown in Figs. 1-6. In these figures predicted values are shown to be very near the heavy lines representing the observed mean driving record. For all projects

(except Project B, as previously mentioned) predictions are within the maximum and minimum bounds observed.

CONCLUSIONS

On the basis of case studies it has been shown that where soil conditions are reasonably well described and adequate numbers of driving records are available, the mean penetration resistance of piling to be driven may be reliably estimated. These estimates are best based on calculations using historical records as opposed to analysis using estimated soil bearing capacity. Further, we have shown that these estimates are best based on wave equation analysis, which is better representative of the mechanics of pile driving, than on more conventional dynamic formulas, such as the Hiley or Engineering News.

The reliability of the method using driving records has been sufficiently demonstrated to allow its adoption in engineering practice. By careful selection of typical driving records from a number of past projects in which a variety of pile and soil types were encountered, a set of standard capacity (driving resistance) versus depth curves could be developed. To make a prediction for any other site one would then need only select his driving formula, assume that the capacity represented by the curve is the same as that computed by that formula, and proceed to calculate the penetration resistance at that depth. This would serve as a valuable tool for both length estimation and hammer selection.

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OPTIMUM WALL INSULATION FOR COLD CLIMATES

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The purpose of this paper is to investigate the optimum thickness of insulation in colder climates where heating requirements dominate the energy requirements for thermal space conditioning.

Optimization is based on cost considerations. Those considerations include the cost of the insulation, cost of installation, and the cost of heating, and may be expressed in equation form as

Total cost = cost of insulation + annual cost of heating + cost of installation

It will be observed that the preceding expression contains a mix of present costs and annual costs. To be meaningful all costs must be either present costs or annual costs. Rather arbitrarily, this paper will convert present cost to annual costs.

The cost of the insulation is converted to an annual cost by multiplying the present cost by a capital recovery factor, C_R . To determine the appropriate capital recovery factor, the life of the facility and the time value of money (or minimum attractive rate of return) must be established. If the life is n years and the required interest rate is i , the capital recovery factor is (from any basic text in Engineering Economy)

$$C_R = \frac{i(1+i)^n}{(1+i)^n - 1} \dots \dots \dots (1)$$

The present cost for the insulation may be expressed as ax , in which x = the thickness, in inches; and a = the cost for 1 sq ft (0.0929 m²), 1 in. (25.4 mm) thick. This expression is very close to market conditions for styrofoam and similar board insulation. For fiberglass and other batt insulation, it is not quite so close to a straight line variation but the difference does not present a major problem, especially since available thicknesses are limited for all types of insulation except spray-on and poured types. On a practical basis, one will

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