Predicting pile driveability: Heather as an illustration of the "friction fatigue" theory

by EDWARD P. HEEREMA*

IN THE series of Papers on the Heather Field pile foundation, this Paper presents the background of the driveability prediction made by Heerema Engineering Service. The prediction was made according to the "friction fatigue" theory, which is based on the assumption that skin friction in clay is gradually lost along the pile shaft as driving proceeds.

A laboratory test is described which illustrates this phenomenon.

A mathematical model is given for the shape of the horizontal stress distribution along the pile shaft; parameters needed in this formulation have been empirically determined, partly from laboratory tests and partly from computer post-analyses of driving experiences. This was necessary as it is as yet not possible to quantify horizontal soil stresses acting on the pile wall during driving on an analytical basis.

The friction fatigue theory presented here proves to be consistent for all pile driving experience in stiff and hard clay in the North Sea investigated by the author, and it is believed, therefore, that it will in general lead to improved pile driving predictions as compared to other available methods.

**Introduction**

The pile driving at Heather Field proved to be one of the most interesting piling operations of the past few years in the North Sea. The soil consisted of very heavily overconsolidated clay at shallow depths, with undrained shear strength values up to 800kN/m², twice the values encountered so far in the North Sea.

At depths near to design penetration of the piles, however, the clay had rather common shear strength values.

It was therefore difficult to make a reliable prediction of the driveability. By conventional means of predicting (i.e. through the use of ultimate static bearing capacity calculations), it was to be expected that it was impossible to drive to the required penetration of 46m with the hammers available.

The author's research group was nevertheless optimistic about the possibility of reaching design penetration. Their predic- tion was that it should be possible to reach design penetration with the second largest hammer available, the Menck 8000. This proved to be correct. Although the blow count was considerably higher than expected over most of the piles' penetration depth, the expected blow count range near design penetration was correct. (Fig. 1: discussed in a later section of this Paper, under "the Heather prediction").

This optimism about pile driveability was based on the recognition of a most interesting phenomenon observed a few years earlier. During the driving in 1974 of a series of 24in dia, test piles by the author's research group to a penetration of up to 23m in stiff clay at Kontich, Belgium, it was noted that although the clay's shear strength gradually increased with depth and therefore the cumulative driving resistance should be expected to increase progressively, the blow count indicated through computer post-analysis that the total driving resistance increased only less-than-linearly with depth of penetration. (If the clay's shear strength were constant, the total wall friction should be expected to increase linearly). The less-than-linear increase of total driving resistance would have meant that the unit skin friction decreased with depth, although the clay's shear strength increased. Of

---


This Paper was first presented (as Paper EUR 50) at the European offshore Petroleum Conference held in London in October 1978, and is reproduced here by kind permission of the organiser.

It is the fifth of six Papers related to the Heather Platform piling, the first two having been published in our November 1979 issue, the third in January this year and the fourth in March.

---

![Fig. 1](predicted-blow-count-curve-and-observed-results.png)

**Fig. 1.** Predicted blow count curve and observed results (see discussion in the text under the heading "The Heather prediction"). Pile, 60in dia. x 23in wall thickness, with shoe; hammer, Menck 8000 (868 000ft. lb)

![Fig. 2](suggested-schematic-unit-skin-friction-profiles.png)

**Fig. 2.** Suggested schematic unit skin friction profiles at four penetrations in a soil with constant shear strength over depth
course, this would be an illogical conclusion. A more sound interpretation of the observed behaviour is then to conclude that somehow skin friction was lost as the pile progressed.

If friction is lost during driving, it is likely that the friction loss is caused by wear of the pile tip. Then it is logical to assume that the clay exerts a peak friction on the pile wall near the tip, where the soil is least disturbed; and to assume that, while the pile progresses downwards, the soil gradually loses its frictional effect on the pile wall.

Fig. 2 shows schematically four unit skin friction profiles versus depth according to this suggestion. It is clear that the cumulative side friction increases less-than-linearly, which would be in agreement with the Kontich test pile results.

The common methods being used to determine friction resistance during driving adopt the implicit assumption that all friction is lost at the pile tip. However, as the pile wall picks up at the tip remains constant while the pile continues to penetrate.

The author has named the phenomenon that friction decreases during driving as “friction fatigue”. After the Kontich tests it was reasoned that a mathematical formulation of the friction fatigue phenomenon could be a significant step towards better pile driving predictions.

Laboratory tests
To obtain more insight into the fatigue phenomenon, a simple laboratory test was devised in which the behaviour of a pile wall in soil could be simulated.

Fig. 3 shows the apparatus used. Soil specimens in the original steel sampling tubes, cut to a length of approximately 10cm, were loaded from above and from below by means of plates applying a vertical stress to the soil. Through a narrow slot in the top plate, a roughened steel blade (12mm wide and 1.5mm thick) was forced into the soil by a hydraulic actuator. The blade was connected to the actuator through a strain gauge and a displacement gauge, so that all forces and displacements could be continuously recorded. After penetrating the blade into the sample, a cyclic displacement was imposed on the blade with (somewhat arbitrarily) an amplitude of 12.5mm and a frequency of 1.6 cycles per second. The cyclic motion was a fair simulation of the continuously reversing slip action of an actual segment of pile wall along the soil during driving.

This test showed that the initial friction of the cyclically moving blade was high, but decreased rapidly. Fig. 4 shows a typical test result. The friction decreases until a certain low residual value is reached.

Formulation of the “friction fatigue” theory
However, interesting this test result might be, it illustrates the “friction fatigue” phenomenon, it cannot be used quantitatively to determine the unit skin friction profile of a pile in the soil from which the sample was recovered.

It must be recognised that if there is a friction decrease along the pile wall, this has to be a consequence of a horizontal soil stress decrease and cannot be caused by remoulding only. Thus it is necessary to think of “friction fatigue” in terms of gradual horizontal soil stress decrease during driving. This decrease may be explained as being caused mainly by irregularities of the pile wall pushing the soil outwards and taking some soil downwards, and by elastic expansion and transverse vibration of the pile, pushing the soil outwards. The soil is thus believed to form a temporary horizontal arch around the pile.

It is not possible to determine the in-situ horizontal stress at any place in the soil analytically: the horizontal stress that the soil exerts on the pile wall is probably even less accessible to an analytical approach. And by no means can one expect that the horizontal stress the clay exerts on the blade inserted in the sample is directly related to the horizontal stress the clay exerts on the actual pile wall in situ.

The inability to quantify the horizontal stress acting on the pile wall analytically and the lack of correspondence between the horizontal stress state in the tested sample and at the actual pile wall have led to the conclusion that it is then necessary to evaluate horizontal stress distributions indirectly from pile driving experience.

By wave-equation 2 computer post-analysis of recorded blow count values, one can determine total driving resistance values. It is however not possible to determine the wave equation is distributed along the pile shaft by these post-analyses. Therefore, an assumption has to be made of the shape of the unit skin friction profile. The peak value should be assumed to be at the pile tip (at zero oscillations), and the frictional stress of the pile can be thought to lie along the horizontal axis of Fig. 4.

As stated earlier, one must think of fatigue in terms of horizontal stress decrease. Therefore, horizontal stress profiles should be determined, although in principle the calculation method could also be developed in terms of “static unit skin friction during driving”, which is more conventional. In an earlier issue of this journal, the author develops an expression of dynamic unit skin friction as a function of horizontal stress, pile wall velocity, and cohesive shear strength. Through this expression, which can easily be made part of the wave equation computer program, it is possible to post-analyse values of horizontal stress (not the distribution, but the total value of the integrated horizontal stress over the pile wall area). The referenced expression is an empirical one which makes use of the horizontal stress to the power 0.7; therefore, in the following the term $\sigma_{h}^{b}$ is considered in preference to $\sigma_{h}$.

Fig. 5 shows the assumed exponential function for $\sigma_{h}^{b}$, in which for simplicity of thought $\sigma_{h}^{b}$ can be substituted by unit skin friction. If we assume:

$$\sigma_{h}^{b} = m \cdot e^{nd}, \text{ and for } d = p: \sigma_{h}^{b} = \sigma_{h}^{b}.$$  \(\text{then }\)

$$\sigma_{h}^{b} = \sigma_{h}^{b} \cdot e^{nd/p}.$$  \(\text{... (1)}\)

In this representation, $n$ is the factor which influences the shape of the $\sigma_{h}^{b}$ curve — the rate of friction decrease. It can be chosen to be solely dependent on the clay’s shear strength and the pile’s penetration depth, expressed as:

$$n = f \left( \frac{c_{u}}{\rho} \right)$$  \(\text{... (2)}\)

which relationship must be determined by}
systematic post-analysis of pile driving field data.

If \( \sigma_{ki}^{h} \) would have to be determined by post-analysis as well, there would be too many unknowns. Fortunately, this value can be derived fairly reliably from the laboratory tests described in the section entitled laboratory tests. For the laboratory test had served only to suggest a shape for the unit skin friction profile, since the stress conditions in the sample could not be considered to be comparable to actual pile driving stress conditions.

However, the initial friction of the blade (during first oscillation) appeared not to be noticeably dependent on the superimposed load of the lever arm — only on the cohesive shear strength value of the clay. Recovery depth of the sample, for example, did not seem to influence the initial frictional value either. This initial value would be predominantly determined by the cohesive shear strength is not surprising, and enables us to determine a relationship:

\[
\sigma_{ki}^{h} = f(c_{n}) \quad \ldots (3)
\]

by the use of the friction/horizontal stress/velocity/shear strength expression from Hoekema, 1970. Test measurements are shown in Fig. 6 whereas the functional relationship between \( c_{n} \) and \( \sigma_{ki}^{h} \) is indicated in Fig. 7.

A linear fit of the test data is given by:

\[
\sigma_{ki}^{h} = 0.25 c_{n} \quad \ldots (4)
\]

\( (\sigma_{0} \text{ and } c_{n} \text{ in } kN/m^{2}; 0.25 \text{ takes into account units of stress } - m^{-1}, \text{ which must be noted in case of conversion of units}) \)

The scatter is very large, but this is, however, customary in soil tests.

It should be noted that the friction values measured on the samples with the highest shear strengths seem to be relatively low, which may be caused by the fact that the most highly overconsolidated samples tend to be most susceptible to strength loss due to the high negative pore pressures after recovery.

As long as it is fundamentally correct, a possible inaccuracy in eqn. 4 will not harm predictions made with the mathematical model being developed, since the post-analysis of pile driving field data leads to figures that work in practice, compensating for such inaccuracies.

As mentioned before, computer post-analysis of a recorded blow count value yields a value of integrated horizontal stress over the pile wall area (comparable to total driving friction resistance). The integration of expression 1 is:

\[
A = \int_{0}^{\sigma_{0}^{h}} \frac{\sigma_{ki}^{h} \cdot \sigma_{ki}^{h} \cdot d(d)}{n} \quad \ldots (5)
\]

For example, in Fig. 8 this "area" \( A \) is then:

\[
A = \left\{ \frac{e^{-ed_{d} - p} + e^{-ed_{d} - p}}{e^{-ed_{d} - p} - e^{-ed_{d} - p}} \right\} \quad \ldots (6)
\]

In fact expression 6 can only be used if the shear strength is constant over the depth of penetration, as only then is there one value of \( n \). If the shear strength profile is complex, a "representative" shear strength value should be taken.

In making predictions, when relationship 2 is considered known, \( A \) must be calculated by taking as many intervals as the complexity of the \( c_{n} \) profile requires; or by calculating \( \sigma_{ki}^{h} \) at many depths and then summing:

\[
A = \sum \sigma_{ki}^{h} \cdot \Delta d \quad \ldots (7)
\]

Each interval has its own \( n \) value, which is derived from relationship 2. Obviously the calculation of \( A \) according to eqns. 7, 1, 4 and 2 can best be carried out by computer.

**Determination of the relationship**

\( n = f(c_{n}, p) \)

To develop relationship 2, it was necessary to post-analyse as many pile driving cases in clay at as many significant penetrations as possible, to obtain a formulation which is reliable because of the large amount of data which has built it up.

"Significant" penetrations were those at which the friction in clay was a large percentage of the total resistance, so that sand friction assumptions did not influence the results too heavily. The pile tip had to be located sufficiently far beyond a sand layer which had been penetrated. Furthermore, the analysed depth had to be sufficiently far beyond a stop to ensure that friction set-up had been lost.

For each post-analysis case and each pile penetration selected, a value of \( A \) was found from the wave-equation analysis. Then a "representative" \( c_{n} \) value had to be chosen, and this was a "weighted" average of the \( c_{n} \) profile, placing more emphasis on those \( c_{n} \) values near the pile tip, as it is there that the main part of the friction is acting. This representative \( c_{n} \) value was needed in eqn. 4 to determine the value of \( \sigma_{ki}^{h} \) for eqn. 5 which in turn yielded the representative value of \( n \) for that specific case.

This value of \( n \) was one of the many points on the graph of \( n \) vs. \( c_{n} \) and the graph of \( n \) vs. \( p \), from which relationship 2 was to be deduced.

Once a preliminary form of relationship 2 had been found in this manner, a more accurate final form of relationship 2 could be determined through a systematic computation of representative \( c_{n} \) values, which substituted the earlier, estimated representative \( c_{n} \) values. This systematic computation (described below) ensured that representative \( c_{n} \) values, which replaced the earlier, estimated representative \( c_{n} \) values, which were found from the wave-equation analysis.

The systematic computation of \( c_{n} \) was as follows:

1. Determine for each case (location, penetration) the theoretical value of \( A \) (eqn. 1) by means of eqns. 7, 1, 4, and the preliminary form of eqn. 2 using the actual \( c_{n} \) profile.

2. Eqn. 5 and the preliminary form of relationship 2 are two equations with two unknowns \( (n_{ki}, c_{n}) \), as \( A_{ki} \) is now an entity known from step 1. Thus \( c_{n} \) can be determined.

With the new \( c_{n} \) values, the \( n \) values were to be re-determined (as \( \sigma_{ki}^{h} \) is changed by \( c_{n} \)) from the already post-analysed actual values of \( A \). The graphs of \( n \) vs. \( c_{n} \) and \( n \) vs. \( p \) were then redrawn, from which an accurate final formulation of eqn. 2 could be deduced.

It should be noted that in the calculation process described above, for reasons
of simplicity of explanation, the local friction reduction due to internal driving shoes is left out. Actually the calculation was made not with $A$, but with \( \Sigma (A \times \text{circumference} \times \text{assumed local friction percentage}) \).

The complete procedure of wave-equation computer post-analyses; determination of preliminary $n$ values; determination of the preliminary $n$ formula; systematic re-determination of representative $c_s$ values; and final determination of the corresponding $n$ values, was carried out for a range of penetrations between 6m and 80m at almost all the locations in the North Sea where piles have been driven through significant clay layers. These locations are: Auk, Brent A, Claymore, Forties FA and FD, Heather, K11 and K13 (Dutch Shelf), Montrose and Ninian Southern. The Kontich test piles were also included. West Solo was left out as the soil data were considered unreliable. Figs. 9 and 10 are the final graphs of $n$ vs. $c_s$, and $n$ vs. $p$, respectively, showing all the post-analysed points as well as the formulation which was developed from them:

\[
n = 0.033 + (0.057 - 1.02 p^{-0.5}) \ln \frac{790}{c_s^{0.4}} \quad \text{... (8)}
\]

Notes:

(1) Range of validity (experience range):

- $70 < c_s < 750kN/m^2$
- $6 < p < 80m$

(2) $0.033$ & $0.057$ have units of $(\text{length})^{-1}$;

- $1.02$ has units of $(\text{length})^{0.5}$

- $790$ has units of stress, which must be noted in case of conversion of units.

(3) Eqn. 8 is the final form of eqn. 2; it will only be modified slightly as more pile driving data becomes available in the future. Indeed, a slightly more emphasised fatigue behaviour (flattening of the lines in Fig. 10) seems justified since first publication of this Paper. Furthermore, recent experience shows that parameters other than those used in this Paper can also influence the magnitude of driving resistance. Relationships found in this respect will be discussed in future publications.

The scattering of the $n$ values around the averaged lines that represent expression 8 indicates that the separate cases do not accurately correspond to the formulated fatigue relationship. But even if a perfect formulation could be found, there would be many discrepancies — caused by a series of factors such as: the uncertainty of the hammer efficiency from case to case; shortcomings of the wave-equation computer program due to the various mathematical simplifications made to it; and, in particular, uncertainty about the correct model of the cohesive shear strength profiles. The many shear strength measurements in one boring profile usually show wide scatters at every level, so that it is often difficult to draw a representative average. Still more uncertain is whether an available boring profile can be considered to be representative for the recorded driving results.

In Fig. 9, an increase of the given shear strength profile of a location would mean an increase of $c_s$; to achieve the same dynamic resistance post-analysed for that case, a higher degree of fatigue will be found, i.e. a larger value of $n$. It can thus be seen that the place of a point in the graph is sensitive to the selected form of the cohesive shear strength profile which, of course, would be the case for any method that associates resistance with cohesive shear strength.

Figs. 9 and 10 exhibit the following interesting phenomena:

-The larger the clay’s shear strength, the smaller the value of $n$, i.e. the lesser the rate of friction fatigue.

This is in agreement with the results of the laboratory tests described in the section on laboratory tests: in harder clay samples the relative friction reduction was found to be smaller.

-The larger the pile’s penetration, the smaller the value of $n$, i.e. the lesser the rate of friction fatigue. This, also, is in agreement with the laboratory test results: if a greater vertical stress was imposed on the sample, resulting in a larger horizontal stress on the blade, the friction reduction was less extreme.
Blow count predictions

To make blow count predictions, horizontal stress profiles can now be determined using expressions 7, 1, 4 and 8, which can easily be written into a small computer program. With these profiles, wave equation computer runs can be made. The following assumptions must be used in the wave equation program to obtain results that agree with the post-analyses that have led to expression 8:

— Hammer efficiency of a single-acting drop hammer (e.g. Menck steam hammer), working at a nearly full drop height and with a cushion block in average condition: 70%.

— Elasticity modulus of bongossi hardwood cushion: 2 000 000 kN/m².

— Coefficient of restitution of bongossi cushion: 0.75.

— Point and side quake: 4mm.

— "Damping" relationships: according to 1.

— Inside friction = outside friction in clay, unless internal driving shoes are used — in which case it is usually assumed that inside friction above the shoe = 50% of outside friction.

— It is assumed that piles do not plug during driving.

It should be noted that different wave-equation computer programs may lead to differences in blow count results.

Pile plugging

It is necessary to explain here why all analyses have been made with non-plugging piles during driving. It is the author's opinion that pile plugging during driving has little to do with pile plugging in the bearing situation. Generally a pile will sooner be plugged in bearing than during driving.

To explain this phenomenon, the hammer impact induces a stress and displacement wave in the pile. As it travels down, it creates a dynamic friction on the inside soil column, so that a stress and displacement wave is also induced in the soil column. The displacement wave in the steel pile, however, travels much more rapidly than the wave in the soil column, due to the great difference between the elasticity moduli of steel and saturated soil. The consequence is that the pile wall shoots past the inside soil column; the inside soil column lags behind. Only after the steel pile tip has reached more or less its final set does the soil column determine its own final set dependent on the resistance it encounters at the tip of the pile. In practice, this is almost always only a smaller part of the set of the pile tip itself; so one can only speak of "partial" plugging.

The static plugging equilibrium is quite a different condition. It is very simply the question as to which is the smaller — the accumulated inside friction, or the resistance of the soil below the cross-sectional area of the soil column.

It is thus clear that plugging during driving and plugging in the static bearing condition are two quite different phenomena, and that a pile will sooner plug in the static bearing condition than during driving.

It is necessary to choose for either a plugged or a non-plugged pile in the wave-equation program. The above reasoning justifies the choice of a non-plugged pile, i.e. making the wave-equation analysis with outside friction, inside friction, and point resistance on the pile annulus.

The Heathen prediction

Expression 8 is an empirical one, based upon field data including those obtained from Heathen. Before the Heathen platform installation, however, no data were available of driving in clay with shear strength profiles beyond 420 kN/m². As Heathen had shear strengths of up to 800 kN/m² the extrapolation that had to be made was enormous. The registered blow count curves (Fig. 1) illustrate that the extrapolation made for the prediction at the time was only partly correct.

There were several additional factors that caused the driveability to be worse than foreseen:

— The hammers generally did not operate at their full drop height; the Menck 8000 was 10% short of drop height on the average. This was due to the inadequate steam supply on the derrick vessel Ocean Builder I. Furthermore the steam temperature was too low, which probably resulted in condensation and hence further reduction in hammer efficiency.

— The platform was originally intended to be set approximately midway between the locations of the borings B2/2a, B4/4a, and B5. In the end it was however set beyond the location of boring B4/4a, where the hardes clay was found. Therefore the shear strength

Fig. 10. "Fatigue factor" n vs. pile penetration
profile of boring B4/4a should be considered determining. The predictions, however, have been made from an average of the three borings.

— There was a difference between the shear strength values determined from the unconfined compression tests, pocket penetrometer tests, torvane tests, and cone penetration tests on one hand and the undrained triaxial tests on the other. On the average, the triaxial tests yielded 23% higher shear strength values than the other tests — which were mutually in good agreement. This difference could not be explained at the time by the soil consultant. We decided to disregard the triaxial tests for the computation of the "expected" blow count curve, which was probably not justified.

In the post-analysis of Heather made later on, the triaxial test (c_u) profile of boring B4/4a was chosen as representative and the assumed hammer efficiency was reduced.

More interesting than looking back at the old prediction is to consider the shape of the theoretical soil friction profile during driving (actually it’s a profile) for Heather at a number of penetrations, calculated by the friction fatigue method using the new figures. Fig. 11 shows these profiles, illustrating clearly how it is possible that although the pile would be expected, from an incorrect conventional approach, to pick up more and more friction as it goes down, the total friction actually does not increase. The “predicted” blow count curve shows that it correlates well with the recorded blow count.

The hammer efficiency has been taken appropriately low. It is, of course, no wonder that the correlation is good, as the experience data themselves have served to build up the general resistance formula; the good correlation merely indicates that the method appears to be consistent.

Note on static bearing capacity
The low average friction along the pile shaft of a deeply-penetrated pile during driving could suggest that also the ultimate static bearing capacity of such a pile would be comparatively low. This is illustrated by the friction-set-up phenomenon, well-known from

Conclusions
(1) The blow count recorded during Heather Field pile driving built up rapidly and then gradually tended to decrease. Unless one accepts the assumption of a plugging pile during driving — a view rejected by the author — this blow count tendency illustrates skin friction loss during driving. This friction loss has been named "friction fatigue".

(2) The "friction fatigue" theory for describing pile driving behaviour in clay, which is described in the text, assumes that the friction experienced by the pile wall is initially large (i.e. at the pile tip), but decreases gradually due to wear of the slip plane along the pile shaft while the pile proceeds downward, as a consequence of the driving action.

(3) The theory thus abandons the implicit assumption in commonly used methods of determining friction resistance during driving, that all friction the pile wall picks up at the tip remains constant.

(4) The gradual decrease of skin friction along the pile shaft as the pile progresses downward cannot be caused solely by clay remoulding; it must be a consequence of horizontal stress decrease of the surrounding soil. This decrease is attributed mainly to the effects of transverse vibration, elastic expansion, and surface irregularities of the pile; the soil is considered to be pushed outwards and taken downwards to some extent. Thus the surrounding soil is believed to gradually form a temporary horizontal arch around the pile.

(5) It is not possible to quantify stress conditions in the soil along the pile shaft during driving in an analytical manner. In order to quantitatively use the "friction fatigue" theory for driveability predictions, it was therefore necessary to complement the theory with experience data, for which a large number of pile driving cases in the North Sea have been analysed.

(6) The assumptions made in the theory, combined with the results of laboratory tests to determine friction in clay samples, and combined with the results of many wave-equation computer post-analyses of driving experiences, lead to the quantitative description of the "friction fatigue" theory through expressions 1, 4 and 8.

(7) The "friction fatigue" theory proves to give consistent results for all pile driving experience in stiff and hard clay in the North Sea investigated by the author; it is believed that it will in general lead to improved pile driving predictions compared with other available methods.

It does not lead to pile driving predictions which are systematically optimistic or pessimistic in comparison to common methods; this depends on the distribution of shear strengths along the soil profile, and therefore differs from case to case.

Nomenclature

A: the integrated value of (horizontal stress)\(^{1/2}\) over the pile’s penetration depth.

\(c_{\text{u}}\): cohesive shear strength of the clay.

\(c_{\text{u}}\): representative \(c_{\text{u}}\) for the resistance profile.

\(d\): depth.

\(i\): initial value before fatigue.

\(n\): "fatigue factor", parameter indicating sensitivity to fatigue.

\(p\): pile penetration.

\(\sigma_{\text{h}}\): horizontal stress of soil against pile wall.

Acknowledgement
The author wishes to express his appreciation for the dedication and enthusiasm with which the soil mechanics team of Heerema Engineering Service has performed the laboratory tests and the many post-analyses needed to develop the described driveability prediction method. Gratitude is also due to Mr. P. J. George for his review of the Paper.

References


