BRIDGE EMBANKMENT DESIGN AND CONSTRUCTION - AN OBSERVATIONAL APPROACH

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Two 13m high surcharged bridge end fills were constructed on soft ground for bridge abutments for the Nicola River Crossing of the Coquihalla highway at Merritt. Site investigation, including penetration vane shear testing, was carried out and stability and settlement analyses performed. Instability and large settlements were anticipated from a conventional filling approach. Therefore, strengthening of the soft soils, using rock-filled shear keys, was adopted and fill placement rates were controlled using instrumentation. The method of Matsuo and Kawamura (1977) was used for construction control. This method is based on using relative vertical and horizontal movements to maintain the factor of safety against failure to realistic but relatively low levels and allows trends towards failure to be identified early. Settlements of up to 930mm were observed, together with 150mm of lateral toe yielding towards the river. The embankments were successfully completed and the experience validated the approach of Matsuo and Kawamura. Long term settlement predictions, including a hyperbolic time-settlement relationship Tan (1971), were used to confirm that piles were not required for support of the bridge abutments.

INTRODUCTION

The Merritt to Kamloops section of the Coquihalla Highway crosses the Nicola River flood plain east of Merritt. The highway design profile and flood considerations required the road pavement to be constructed approximately 9.5m above the flood plain, adjacent to the river bank. The bridge was to have a settlement sensitive continuous span with pile supported piers. However costly pile support for the abutment could be avoided, provided long term bridge end fill settlements could be minimized.

The presence of soft soils required that the bridge-end fills be surcharged to minimize post-construction settlements. However, the fill heights created potential stability problems and very large settlements were predicted. Since there were a number of uncertainties and variations in the ground conditions, an observational approach was adopted with provision for use of abutment piles should stability or long term settlements become unacceptable. To that end, it was decided to reinforce the ground and carefully monitor the fill placement with provision for slowing or halting fill placement. The results of the monitoring would also be used to assess the long term settlements of the bridge structure.

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The ground conditions, analyses and construction techniques are described for the south fill only, since this was more critical than the north side, due to poorer soil conditions.

GROUND CONDITIONS

The bridge site is located on the flood plain of the Nicola River with the bridge end fills close to the river bank. The site investigation (see Figure 1) revealed weak and compressible alluvium overlying thick deposits of relatively incompressible granular inter-glacial deposits overlying very dense glacial till at 45$\pm$ m (see BH-1005 - Figure 2). A range of thicknesses and material types was encountered across the site in the alluvial stratum.

On the south side a maximum total thickness of 5.8m of clay was encountered. The clay contained silt, together with sand partings. The soft clay had shear strengths which varied between 10 kPa and 50 kPa, measured with field vane equipment and in laboratory triaxial compression testing. The results of field vane test V7 are shown in Figure 3. A correction factor of 1.0 was applied to the measured values, based on the work of Bjerrum (1972). The average sensitivity of the clay, as determined from the field vane test results was 6.

Oedometer testing of the more clayey zones indicated values of compression index (C_c) of 0.33 and 0.11 for the upper and lower clays respectively, in DH 1005.

An effective friction angle of 30^o for the silt was determined from triaxial testing, assuming zero effective cohesion values.

ANALYSIS AND DESIGN

Significant variations in ground conditions were encountered and therefore several design profiles were considered for analysis.

Settlements

A surcharge fill thickness of 3m was determined, based on the method of Johnson (1970), thus requiring a fill height of 12.5m for a period of 6 months. Table 1 below presents estimates of settlements for a 9.5m and 12.5m high embankment for two assumed thicknesses of soft clay of 2m and 4m. Reduced thicknesses were used to reflect the proportion of sand in the profile.

Table I - Settlement Estimates

	Settlements (mm)				
Fill Height (m)	During Construction	After Construction		Total	
	Clay Thickness	2m	4m	2m	4m
9.5	150	250	550	400	700
12.5	200	350	700	550	900

These settlements are based on parameters obtained from laboratory testing, in the case of the soft clay, and empirically derived relationships, in the case of the stiff soils and granular materials. A value of C_α = 0.01 was used for long term creep assessment based on the work of Mesri and Godlewski (1977).

The settlement estimates assumed conservative drainage conditions as a "worst" case scenario, although it was appreciated that in practice settlements were likely to occur more rapidly. Consolidation testing in vertical and horizontally oriented samples did not reveal a significant anisotropy.

Stability

The presence of the soft clay created the potential for foundation stability problems during and immediately following till placement. The situation was made worse by the proximity of the river banks. In the long term, after full dissipation of construction pore pressures, the stability was satisfactory, with an estimated minimum factor of safety of 1.5. Friction angles of 38° and 34° were assumed for the compacted and uncompacted fill, respectively.

A number total stress stability analyses were carried out, assuming no foundation drainage, to assess the construction stage stability and the possible need for improvement measures. Table 2 below presents estimated factors of safety based on the method of Janbu (1959) and assuming a wedge failure mechanism sliding on the weak clay layer. "Worst" case and "best" case profiles, A and B respectively (see Figure 3), were developed for use in the stability analyses.

Table 2 - Estimated Factor Of Safety (Profile A)

	Embankment Height			
Boundary Condition	4m	6m	8m	12.5m
No river	1.8	1.3	1.0	0.8
River at 14m from toe	1.6	1.2	0.9	0.7
River at 3m from toe	1.2	1.0	0.8	0.6

The analyses indicated that about 6m of fill may be placed before instability is approached. The analyses assume a "worst case" soil profile (A) but the field work indicated that such conditions do not exist over the entire site. Therefore, a limited analysis for the maximum height of fill (12.5m), using a "best case" soil profile (B) for two boundary conditions was performed. These values are presented in Table 3 below.

Table 3 Estimated Factor of Safety (Profile B)

Boundary Condition	Factor of Safety
No river	1.25
River at 3m from the toe	1.15

The relatively low safety factors in Table 3 also indicate, when taking into account the overall reliability of stability analyses, that even with a "best case" profile the stability of the embankment could be marginal, depending on the rate of porewater pressure dissipation.

Ground Strengthening

Ground reinforcement options for this small and relatively remote site were considered. By analysis, it was determined that the most cost effective and optimum solution was to construct pervious shear keys through the soft clay, to provide reinforcement and drainage.

Two 1m wide rock filled trenches were considered for beneath the end slopes, across the full width of each embankment (see Figure 4). The trenches would be taken down through the upper clay to the underlying sand and gravel. The factors of safety in Table 2 were estimated to increase as indicated in Table 4 below, not taking into account any drainage:

Table 4 Estimated Factor of Safety With Key Trench (Undrained)

Boundary Condition	Embankment Height			
	4m(1)	6m(2)	8m	12.5m
River at 14m from toe	1.6	1.3	1.1	0.9
River at 3m from toe	1.2	1.1	0.9	0.8

(1) failure zone beyond trenches
(2) only one trench in failure zone (interpolated values)

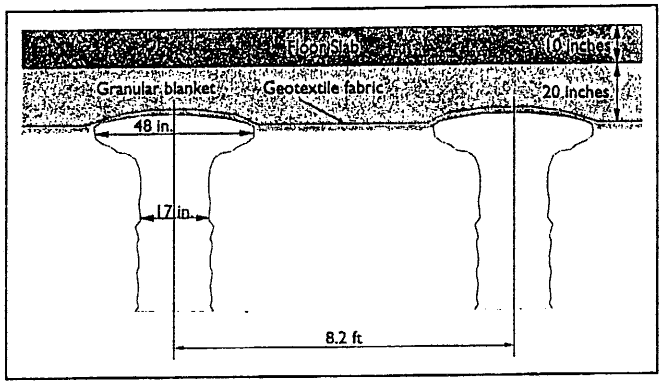

Figure 7: Cross Section of VCC System below Warehouse Slab

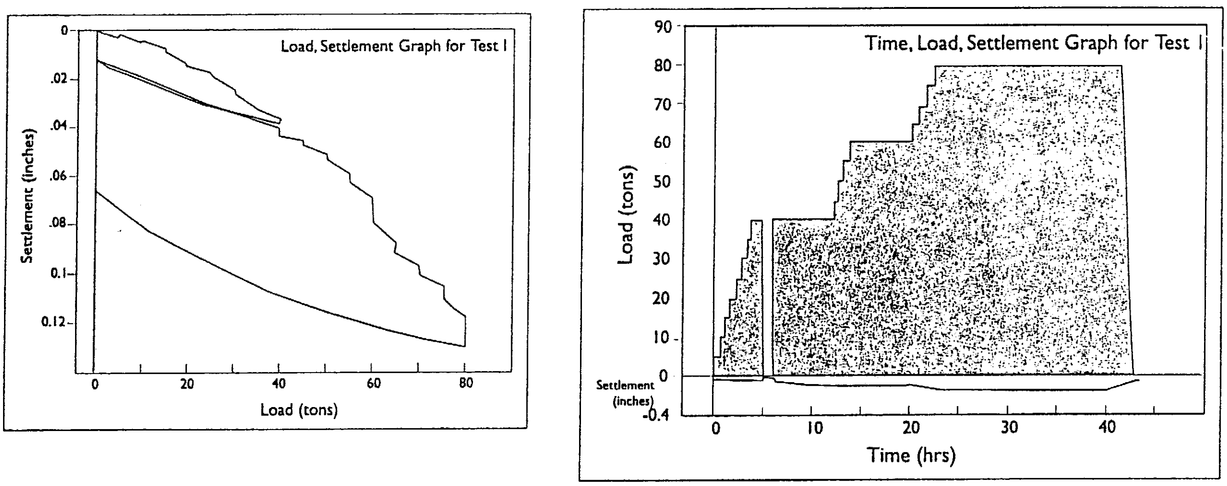

Figure 8: Warehouse VCC Load Test Results

SUMMARY

The Vibro Concrete Column System is a very effective ground improvement technique for large area loads such as embankments and structures with large floor loads. The system uses the load transferring characteristics of piles, while mobilizing the full ground improvement potential and speed of the vibro system. The combined effect produces a displacement foundation system that is not only extremely efficient and quiet, but is also very economical.

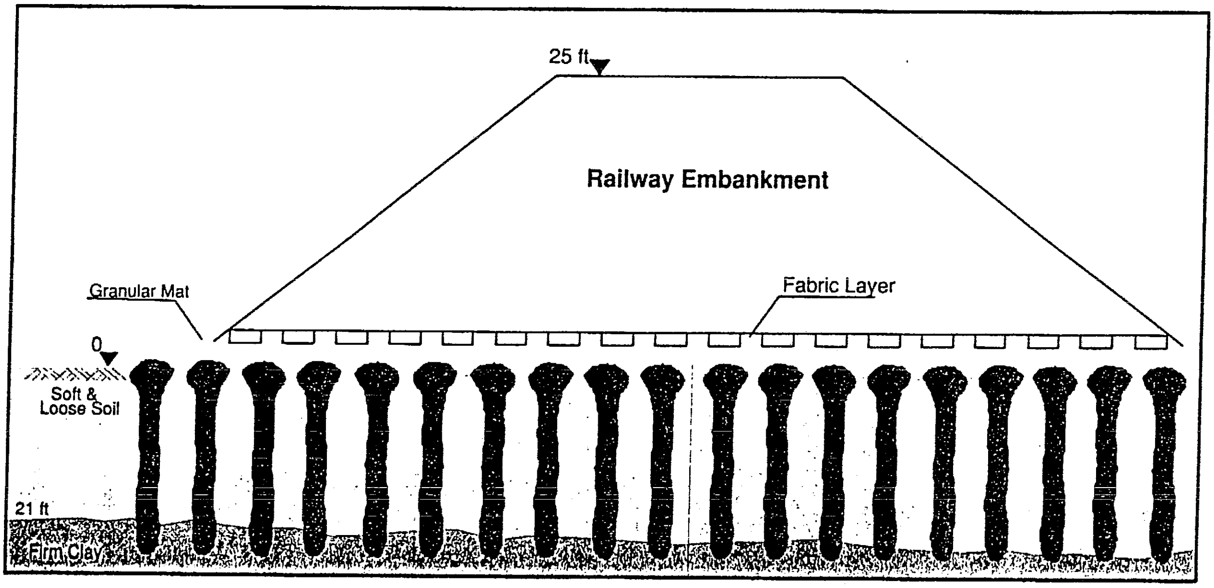

Figure 6: Cross Section of Railway Embankment on VCC System

3. Warehouse Building, Cheddleton, England

A high bay warehouse building with a heavily loaded floor area was to be built on a site underlain by 13 to 30 feet of weak alluvial deposits. The borings encountered loose ash fill overlying soft silty clays, loose silts and bands of peat up to 3 feet thick, overlying dense sands and gravels. The variability of the highly compressible soils gave concern regarding total and differential settlements. The floor slab, designed to support point loadings from the high bay racking system produced a 1100 psf ground bearing pressure. Allowable settlements were 0.6 inches total and 0.4 inches between any two points on the slab (1:1000).

To provide the required bearing pressure, a total of 806 unreinforced Vibro Concrete Columns were constructed to depths of up to 38 feet on an 8.3 foot grid. The columns had a minimum diameter of 17 inches and were formed with an enlarged base in the sands and gravels and with a mushroom head at the ground surface. A 20 inch thick granular blanket was placed over a geo-textile fabric resting on the tops of the concrete columns. The 230 foot by 1230 foot slab was designed as a 10 inch thick slab on grade (see Figure 7). Two load tests were performed using anchor columns as reactions. At 2.5 times capacity the settlement of the VCC was less than 0.2 inches (see Figure 8).

A concrete design strength of 4,000 psi was employed. A full-scale load test on a column was successfully performed to an ultimate load of 200 tons.

2. Railway Embankment, Munich Germany

A new high-speed railway line was planned to be constructed adjacent to an existing railway embankment. Beneath a portion of the alignment, the soil borings encountered a 12-foot thick soft silt and peat layer (see Figure 4). The original proposal consisted of total replacement of the soft soils to prevent long term settlement and stability problems. As the existing adjacent line had to remain in use, this proposal would have necessitated a sheet pile and anchor system to protect the existing embankment. An alternate VCC program was proposed which was more economical and saved time in the schedule.

The Vibro Concrete Columns were designed to carry 50 tons per column and were constructed on 9 foot and 12.8 foot spacing, depending on embankment height. Load tests measured an ultimate capacity in excess of 150 tons and settlements at design load of 0.15 inches (see Figure 5). The concrete columns were covered with a granular mat 1 to 2 feet thick and a fabric layer with a tensile strength of 15 to 30 psi to resist horizontal shear forces prior to raising the embankment (see Figure 6). The predicted settlements were subsequently confirmed on completion.

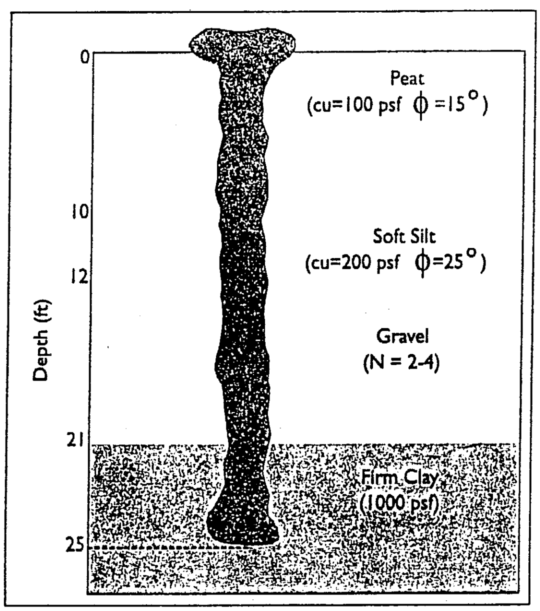

Figure 4: Generalized Subsurface Profile

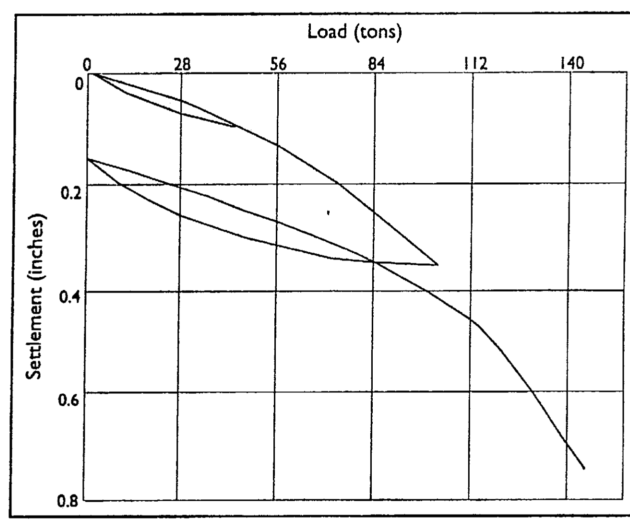

Figure 5: Load Test Results

Testing of the VCC system can also be performed if desired. Load testing of columns can be performed to verify predicted settlements under design loads.

CASE HISTORIES

1. Tank Foundation, Philadelphia, Pennsylvania

A proposed site for a new storage tank was to be built in an existing tank farm. The new tank is 75 feet in diameter and 32 feet high. Assuming the weight of caustic soda at 11.5 lbs/gal, the full tank would impose a load of approximately 2.76 ksf on the foundation soil.

Soil borings indicated that a weak zone of organic silt and peat, approximately 10 to 15 feet thick, was encountered beneath 5 feet of miscellaneous fill and rubble.

Hayward Baker recommended installing Vibro Concrete Columns to transfer the loads through the weak soil strata to bear on the densified sand and gravel (see Figure 3).

The VCC process employs a bottom-feed depth vibrator which penetrated the weak soils to a level of dense material suitable for supporting the load. Concrete was pumped at high pressure through the vibrator assembly to form a high capacity plug or "bulb" of concrete in the underlying granular layers. These layers were also improved by the vibratory action. The vibrator was then slowly withdrawn as concrete was pumped at maintained pressure. A continuous shaft of concrete was formed up to ground level. The vibrator then re-penetrated the top concreted portion of the pile to form an enlarged "mushroom" head. A transfer platform consisting of three layers of geogrid-reinforced granular fill (i.e., 3 to 4 feet) was placed over the VCC columns to spread the load uniformly (see Figure 3).

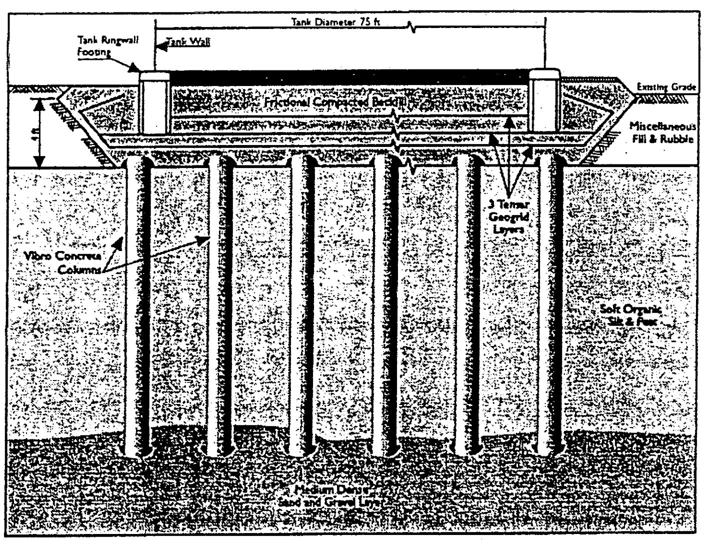

Figure 3: Tank Foundation Cross Section

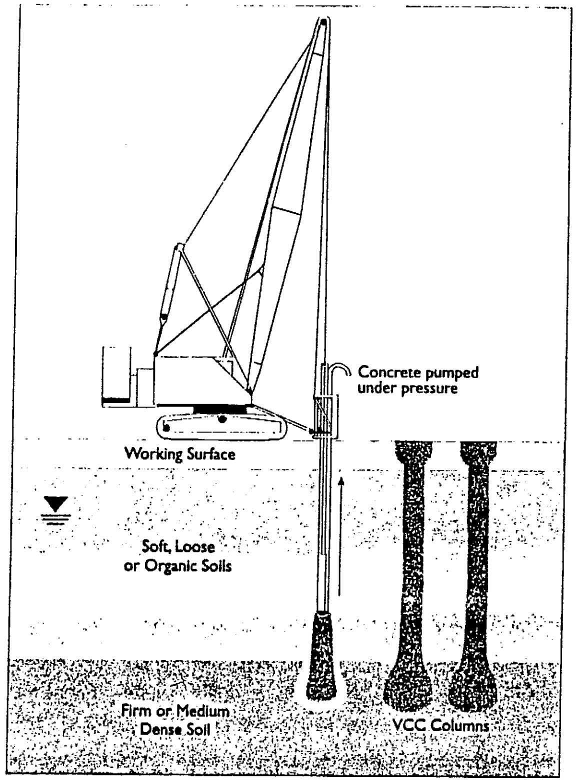

Figure 1: Vibro Concrete Column Construction

MONITORING AND TESTING

The Vibro Concrete Column system can be fully instrumented with an in-cab display to monitor the construction sequence documenting concrete pumping pressure and vibrator power consumption, all related to time and depth. A hard copy printout provides a record of the construction of each Vibro Concrete Column (see Figure 2).

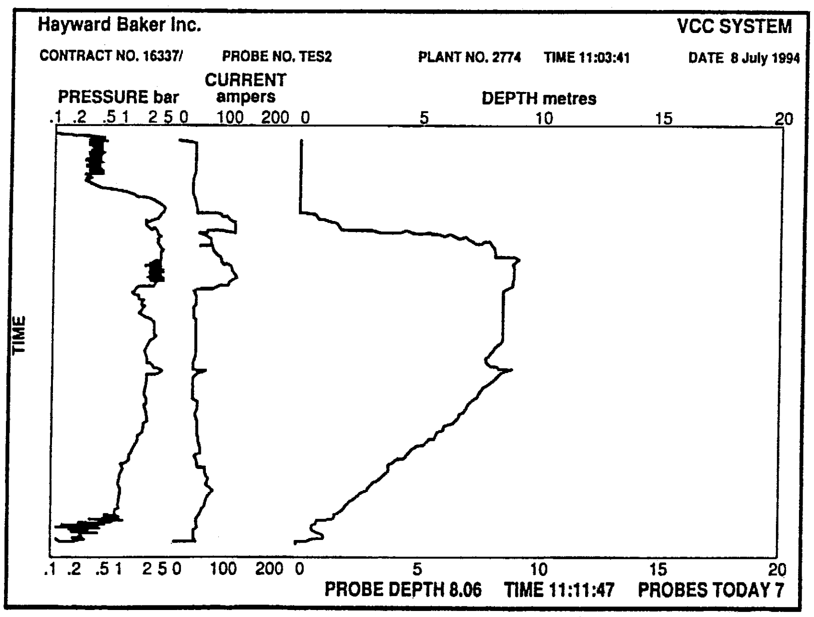

Figure 2: Typical VCC Computer Printout

soft organic clays and silts. The system thus narrows the normally wide gap between piling and ground improvement. The VCC technique combines the ground improvement advantages of the vibro systems with the load carrying characteristics significant floor loads, the VCC technique is used to reduce settlement, increase bearing capacity, and, if necessary, increase slope stability. This vibro-displacement technique will densify granular soils and transfer loads through soft cohesive and organic soils.

The analysis and design of the VCC improved site is essentially the same as would be performed for a pile foundation, except that the improved soil parameters are used. For large areas loads the VCC system can be overlain by a granular mat, sometimes reinforced by a geogrid, to evenly distribute the structural loads to the VCC system. At the ground surface a slight mushrooming of the concrete column occurs which also assists with the load transfer. Therefore, the planned structure can be designed with a standard shallow foundation system or, in the case of embankments, with a uniform bearing pressure. The addition of the granular mat is not needed if a sufficient thickness of surface granular soil is present, which will be densified as a result of the VCC construction and act to distribute the loads to the VCC system.

In general, the Vibro Concrete Columns are used without reinforcement. However, if required, single bars for uplift loads or short cages (<20 ft) can be vibrated into the completed columns for lateral load conditions.

Typical concrete design strengths of 3000 to 4000 psi are employed. The Vibro Concrete Columns develop ultimate loads in excess of 150 tons with safe working loads in the range of 40 to 75 tons.

CONSTRUCTION

The VCC technique makes use of the specially-designed electrically-driven bottom feed vibrator which penetrates the weak subsoils to a level with either sufficient bearing capacity or to a suitable granular layer which can be compacted and improved by the action of the vibrations. During the initial penetration of the vibrator, weak cohesive and organic soils are displaced while granular layers are densified by the vibratory action.

Once improvement of the load bearing formation has been completed, the concrete pump is turned on to introduce high quality concrete from the tip of the vibrator into the ground and the column construction process is started by operating the vibrator to form a bulb of concrete at the base of the concrete column.

The vibrator is then slowly withdrawn, the concrete pressure being maintained, and a continuous shaft of concrete is formed up to ground level (see figure 1). Reinforcement is then added, if needed.

Although the computed factors of safety (undrained) were low, it was felt probable that sufficient drainage would occur to allow the embankments to be built without any delays and the key trench method was adopted.

CONSTRUCTION

Instrumentation

The following instrumentation was installed at the south bridge end fill, at locations indicated on Figure 4:

> Piezometers (4 No.) - a three tube system fitted with a high air entry coors filter was installed in soft or soft to firm clay at depths varying from 1.4m to 3.0m below ground level.
>
> Toe Movement Gauges (7 No.) - the gauges were formed from wooden stakes with a reference pin onto which a steel tape was hooked.
>
> Surface Settlement Gauges (5 No.) - standard extendable piping anchored to a base board.

Monitoring and Performance

Readings were taken very frequently during the early stages of construction and progressively less frequently as knowledge and confidence in the fill behavior accumulated. In the weeks immediately prior to preload removal, settlement readings were increased to twice weekly to aid long-term settlement assessment.

Fill was placed over a 3 month period and left in place for a further 3 months. The settlement-time plots for the bridge end fill are presented on Figure 5. Little settlement occurred until 2m of fill had been placed and then large settlements occurred concurrently with filling. Construction was generally continuous except for two periods, as requested by Klohn Leonoff. Work was stopped for a week at 5m height after 400mm of settlement to allow the general settlement behaviour to be assessed. Work was also stopped at a height of 12m after settlements, of between 650mm and 800mm, started to accelerate, following rapid fill placement. Construction was halted for two weeks and then re-started at a much slower rate. There were also occasional short shutdowns due to inclement weather. Total settlements, at the time of surcharge removal, ranged from 750mm to 930mm and were continuing. The fill was placed to an elevation and, therefore, the total thickness of fill placed was approximately 13.3m.

Stability Assessment

Porewater Pressures

Stability charts were prepared for use in construction monitoring. The key trenches were modeled as a frictional soil layer intersecting the soft alluvium. Analyses were carried out for cases of the river at 3m and at 14m from the toe and also the case of no river. Factors of safety were computed for a series of embankment heights, each subjected to a range of excess porewater pressures in the foundation. The stability chart for the two piezometers showing the greatest responses are presented on drawing Figure 6.

The maximum excess porewater pressure recorded was equivalent to a 4.6m head of water and occurred at an embankment height of 12.3m. Dissipation of excess porewater pressures was rapid and generally occurred as the embankment loading increased. This is considered to be due to the laminated nature of the alluvium and also due to the presence of the key trenches. The factors of safety, indicated by the stability charts, reduced to minimum values of between 1.6 to 1.8. No danger to the embankment was indicated by these values, all being greater than 1.5, but the piezometer readings did not necessarily ensure that failure would not take place along a zone of locally higher porewater pressures not connected to the piezometers. For this reason, performance monitoring also included vertical and horizontal movement gauges.

Ground Movements

Stakes along the toes of the fills were used to monitor horizontal and also vertical movements. The maximum horizontal movements occurred, as expected, where the toe was closest to the river (3m). Maximum movements of 148mm were recorded at gauge #44 as presented on Figure 7 The movement at gauge #43 was also large (100mm). The remainder of the gauge movements were relatively small, though up to 72mm..

At lower embankment heights, up to 6m, the horizontal and vertical movements of gauge #44 were very similar. Thereafter the horizontal movements greatly exceeded the vertical movements by a factor of approximately 2, to the maximum value of 148mm. Movements continued during the two periods in which construction stopped, though the rate decreased slightly with time. The relatively large and accelerating horizontal toe movements were considered to be possible indicator of the onset of failure.

Toe movements alone cannot be quantitatively correlated with the embankment safety and, therefore, the approach of Matsuo and Kawamura (1977) was used as an aid to assessment of the embankment stability. They developed an empirical relationship between the settlement at the centre of an embankment and the horizontal toe movement, at any given time. The relationship is presented in terms of the ratio of the embankment height during construction (P) to the failure height of the embankment (P_f), based on the assessment of many embankments, including ones which failed.

The chart of Matsuo, which is for largely undrained conditions, and data from the Merritt embankment are plotted on Figure 8. The characteristic maximum embankment settlement was 820mm, although the maximum recorded settlement was 930mm. The Merritt data has the characteristic form found by Matsuo and indicated that the embankment was relatively safe, having P/P_f ratio of between 0.7 to 0.8 i.e. the embankment height was at about 75% of the failure height. However, lack of experience with this approach and prudence caused us to halt construction on two occasions.

Matsuo did not publish factors of safety associated with his embankment studies and therefore the P/P_f ratios cannot be directly compared with factors of safety. However, the reciprocal of the P/P_f ratio could be considered analogous to a factor of safety. Since Matsuo acknowledges the curves could be moved to the right for partially drained conditions then a reasonable agreement between his work and the porewater pressure stability charts may be inferred.

LONG-TERM SETTLEMENT ASSESSMENT

The settlements were continuing when the construction schedule required the preload to be removed after 24 weeks. At that time a decision had to be made regarding the use of piled or spread foundations for support of the bridge abutments. The long-term settlements under the preloading conditions were estimated, together with those for the two foundation options.

Three estimates of long term settlements were made. An initial one based on laboratory test data was carried out during the design stage, as summarized in Table 1. Two empirical methods, logarithmic and hyperbolic, based on field performance, were also used to assess post construction settlements. The data used for the empirical solutions was obtained from a "smoothed" plot of the average settlement points. The method of Johnson (1970) was applied to the preload settlements to assess the long term settlements under the lower permanent loadings.

In the logarithmic method the data was plotted conventionally and a linear creep portion of the curve was obtained from 16 weeks onward. A settlement rate of 120mm per log cycle was obtained, as shown on Figure 9.

Tan (1971) and others have demonstrated that soil behaviour is hyperbolic under many loading conditions, including embankment loading. A hyperbola may be transformed into a straight line which is amenable to engineering application and interpretation. A normal settlement (s) versus time (t) plot may be re-plotted in the form of a settlement ratio ($^t/s$) versus time (t), as shown on Figure 10. A characteristic of the plot is that the creep behaviour of the soil is represented by a straight line. The plot before the creep portion represents both primary and secondary consolidation behaviour and the start of the linear portion is the point of zero excess porewater pressure. It may be readily shown that the reciprocal of the slope of the creep portion of the data curve is equal to the ultimate settlement, i.e. the asymptote of the hyperbola. Figure 10 indicates that pure creep started at week 16 and that the ultimate settlement is 875mm.

The three estimates of the average long term settlement of the preload fill are summarized below in Table 5. It is of interest to note that the hyperbolic result is mid-way between the 50 year and 100 year logarithmic values. This, of course, may be coincidence.

Table 5 - Estimated Average Long Term Settlements - Preload Fill

Method	Ultimate Settlement	Residual Settlements (2)
Laboratory	900	80
Logarithmic (1)	855 - 895	35 - 75
Hyperbolic	875	55

(1) 50 years and 100 years
(2) Settlement remaining assuming average settlement of 820mm prior to bridgeconstruction

The estimated range of residual settlements for the preload fill and the two foundation options is shown in Table 6 below, considering only the average and lowest values of preload settlement.

Table 6 - Estimated Residual Settlements

Loading Case	Equivalent Fill Height (m)	Estimated Residual Settlements (mm)
Temporary Preload	3±	40 - 150
Spread Footings	2±(1)	30 - 80
Piled Foundation	0	5±

(1) with respect to the soft clay

The range of settlements above reflects the variation in the predicted settlements and the range of measured settlements prior to construction. Greater weighting was placed on the empirical predictions and on the average of the settlements in predicting post construction settlements of a spread footing foundation. Accordingly, post construction settlements were considered likely to be in the range of 40mm to 50mm

An analysis of the bridge deck behavior found that it could tolerate at least 40mm, and possibly up to 60mm of abutment settlement. The decision was made to adopt spread footings in the bridge end fill and to precisely monitor the relative settlements of the abutments over time to check the actual behavior. Provision for re-leveling by jacking was made in the bridge design.

Although no settlement readings have been taken there is no apparent distress to the bridge deck in the ten years since construction.

CONCLUSIONS

The benefits of the observational approach were, once again, demonstrated by the significant cost saving in avoiding the use of long large diameter piles. The method of embankment construction control of Matsuo and Kawamura was found to be simple, economical and effective and confidence for its future use was obtained. The empirical hyperbolic method of settlement was found to be a useful complement to the more traditional logarithmic approach and has an apparent advantage of yielding a single value for ultimate settlement.

ACKNOWLEDGMENTS

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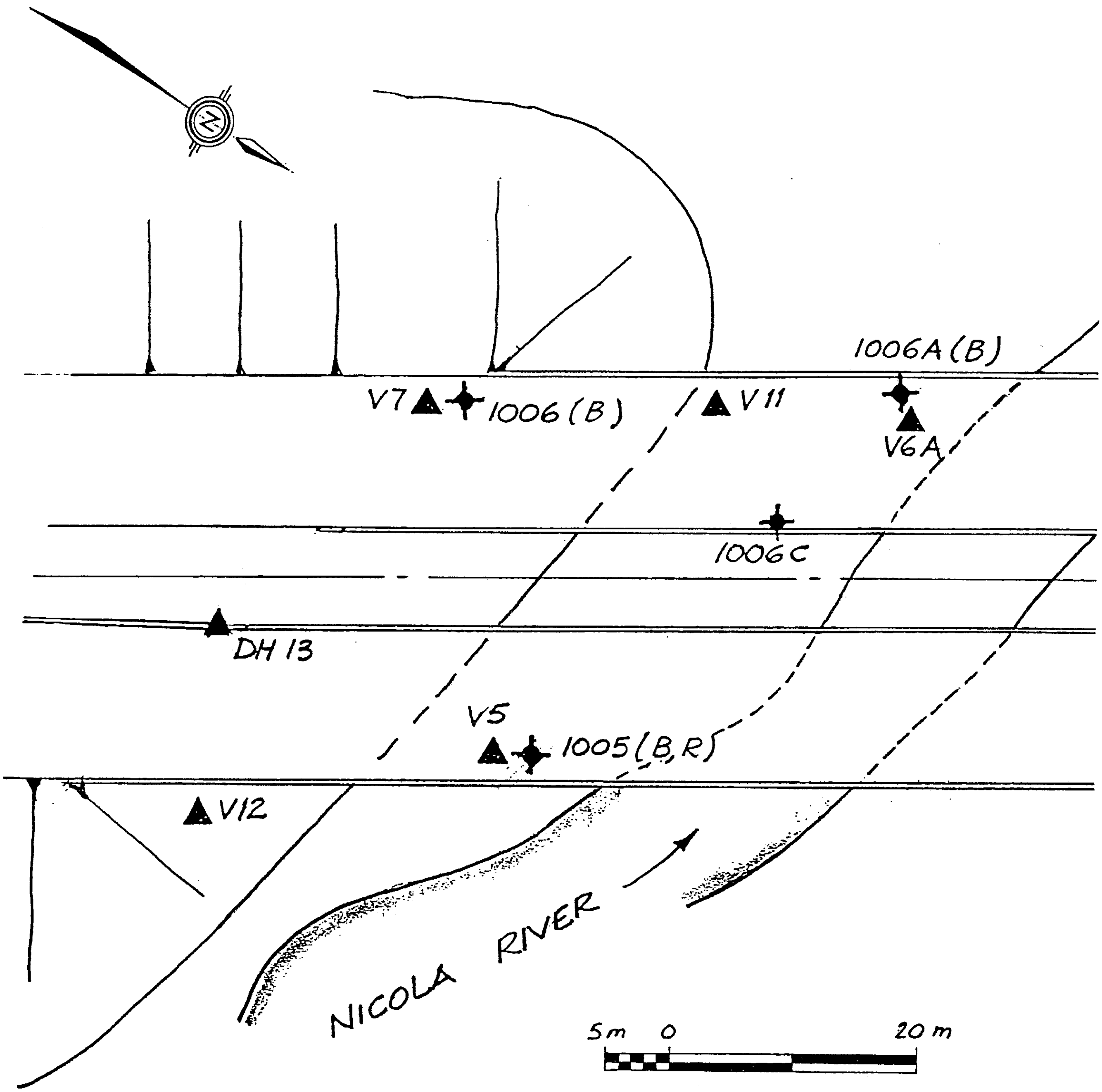

FIGURE 1 SITE AND TEST LOCATION PLAN

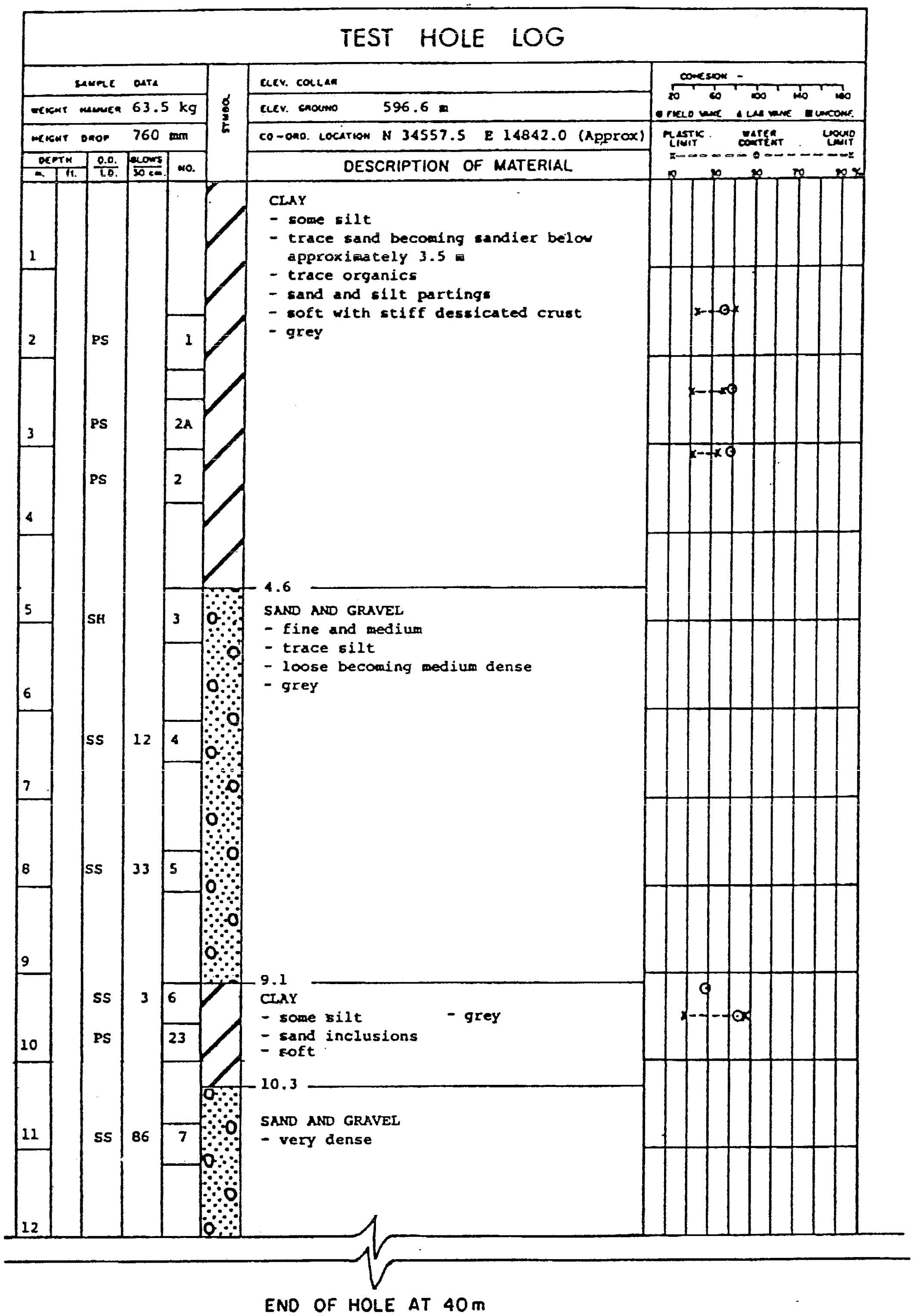

FIGURE 2 TEST HOLE LOG (DH1005)

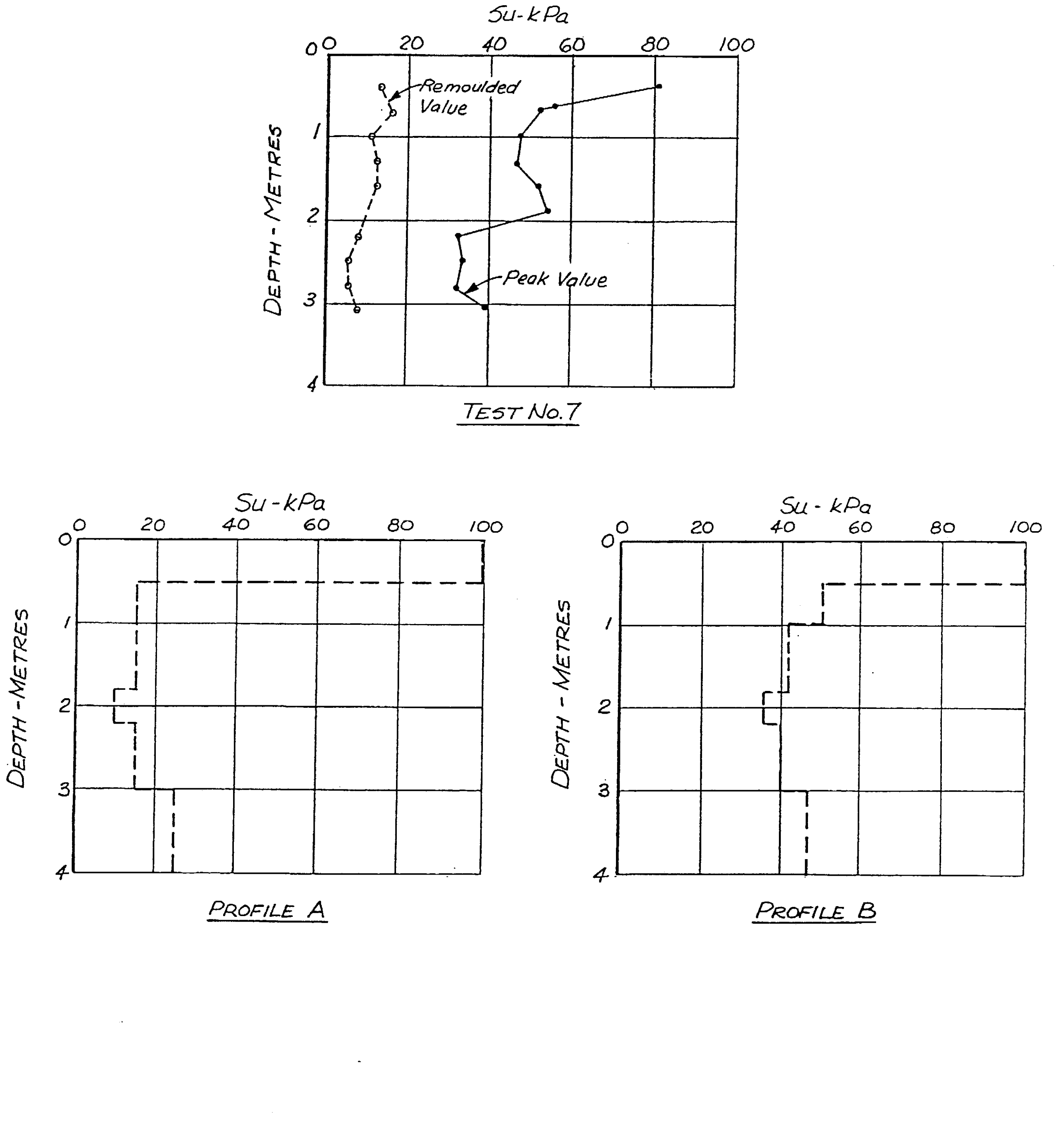

FIGURE 3 SHEAR STRENGTH PROFILES

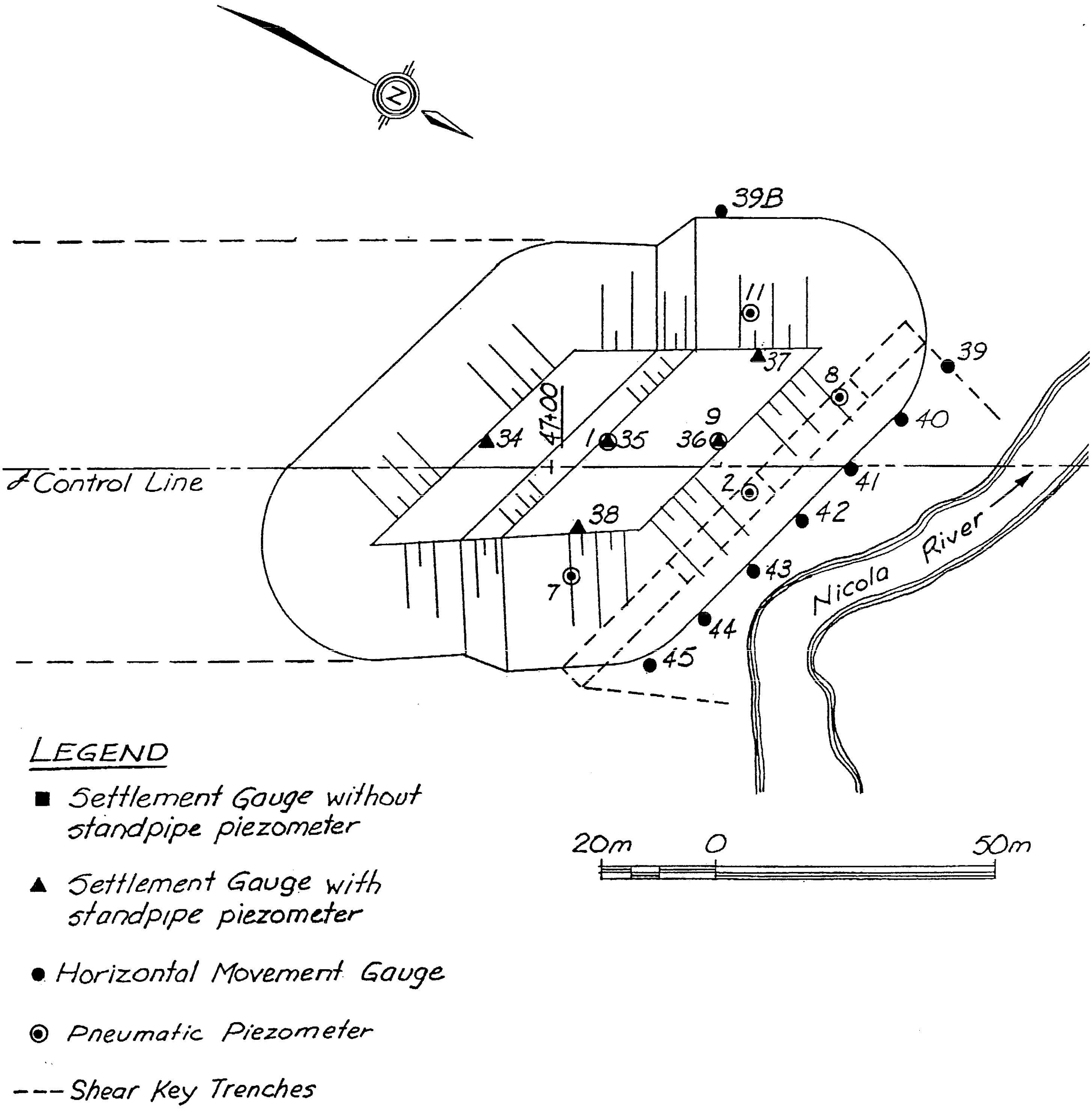

FIGURE 4 LOCATION OF INSTRUMENTATION AND SHEAR KEY TRENCHES

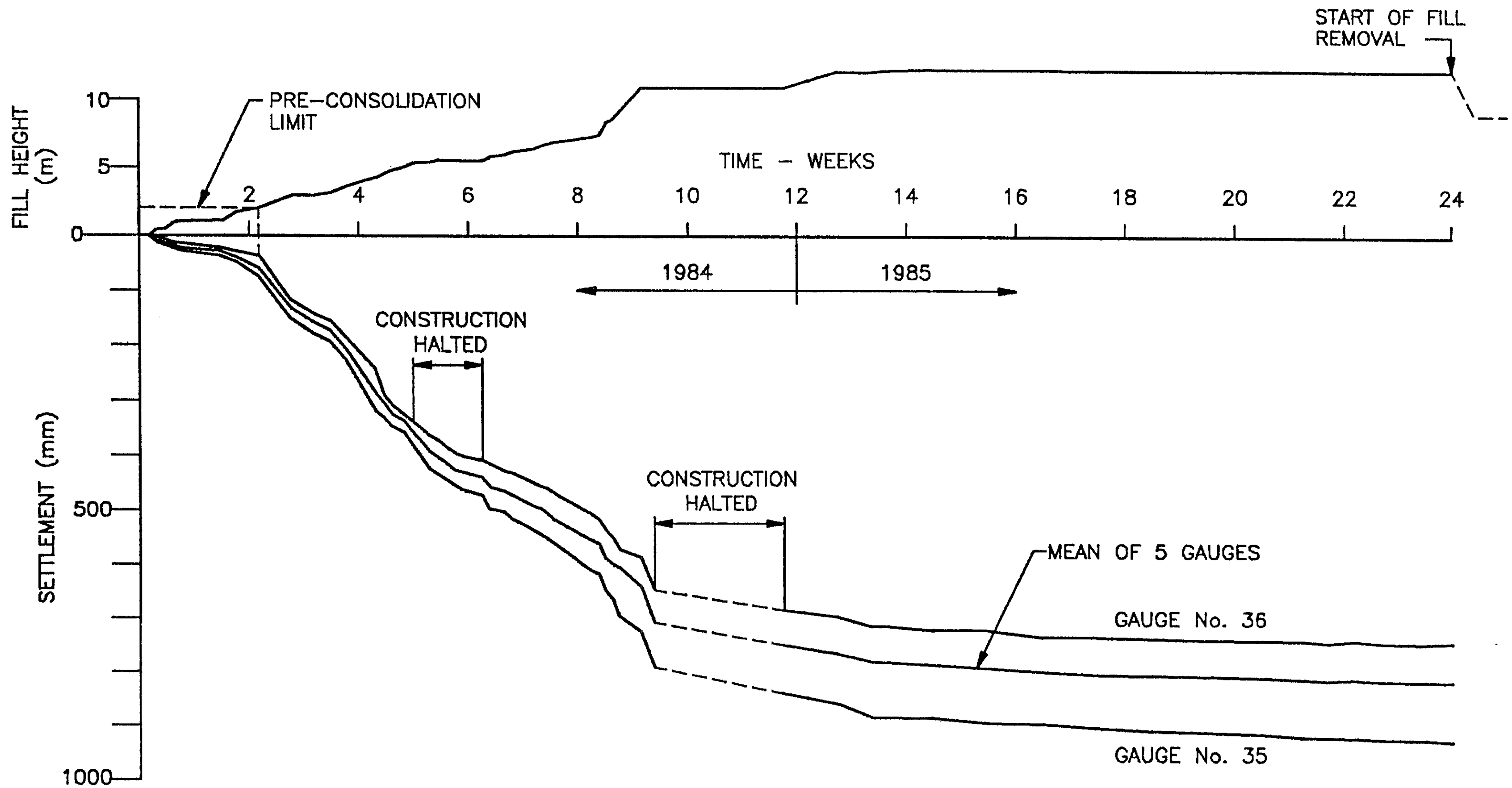

FIGURE 5 SETTLEMENT v TIME

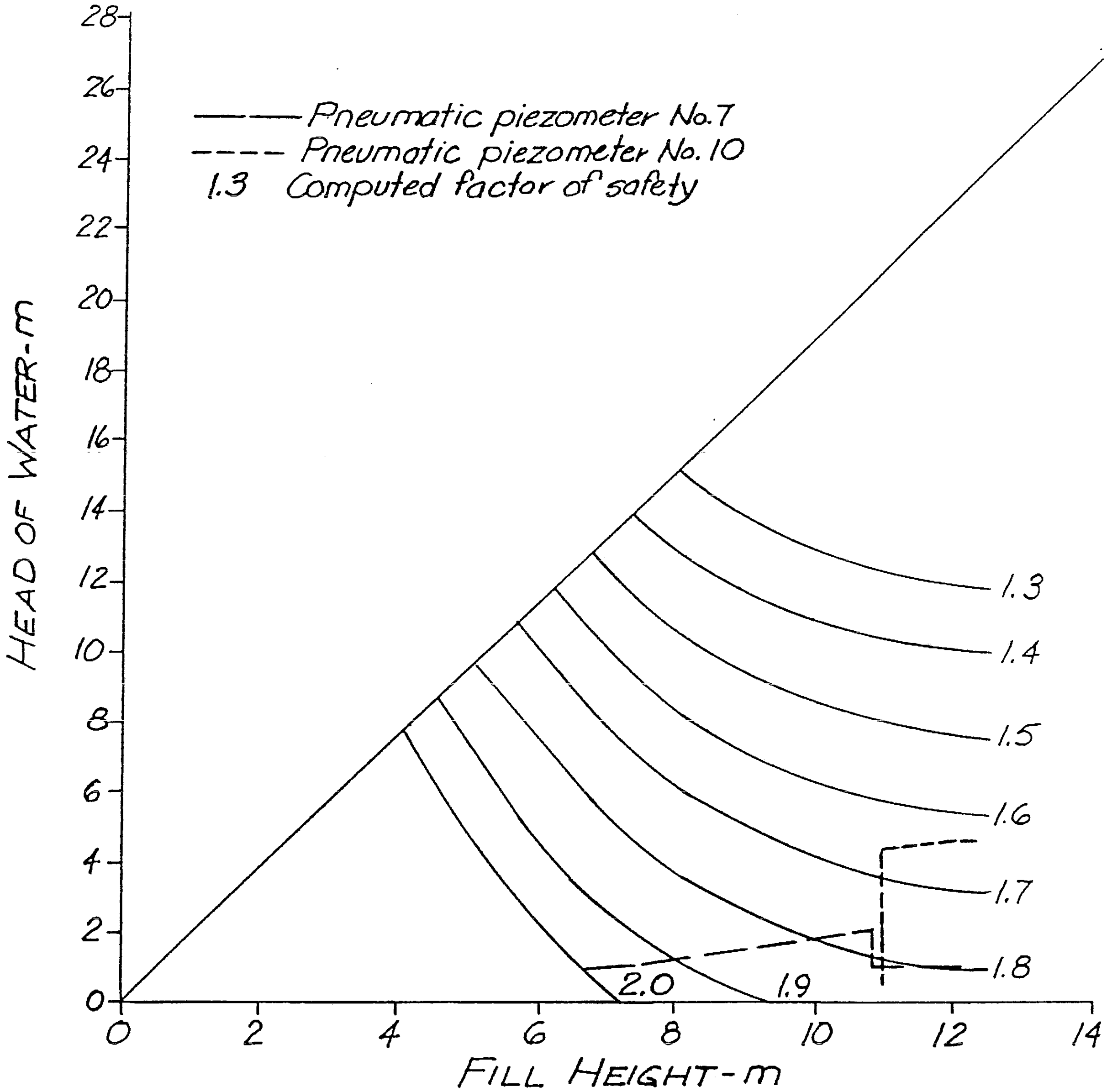

FIGURE 6 STABILITY CHART (POREWATER PRESSURE)

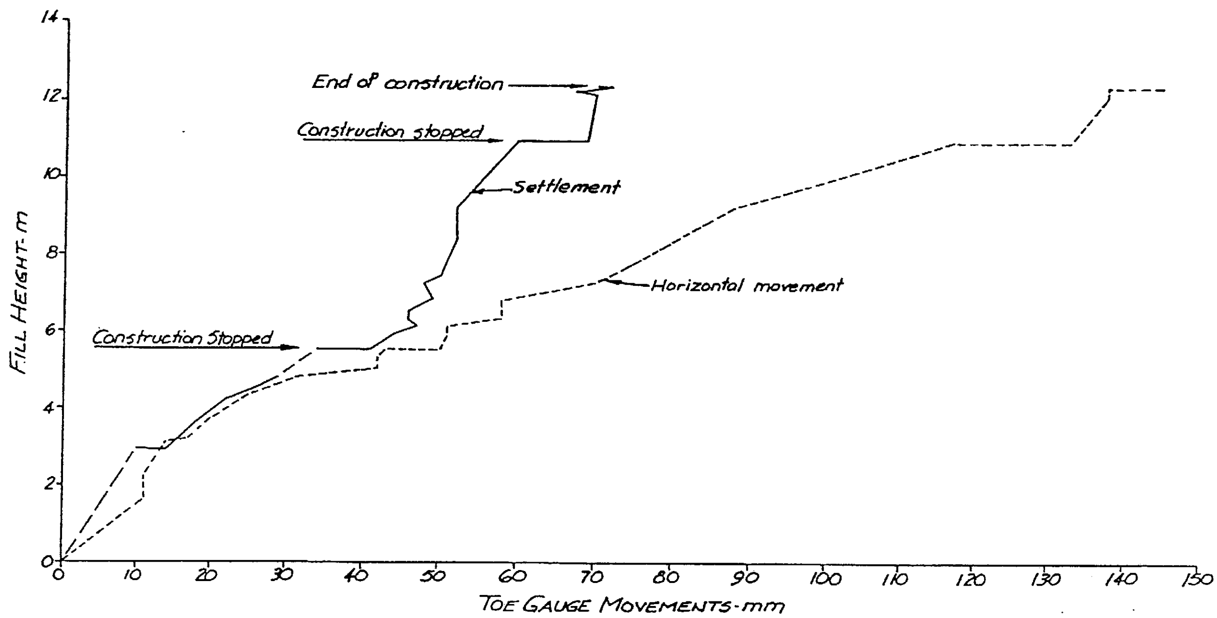

FIGURE 7 TOE GAUGE MOVEMENTS

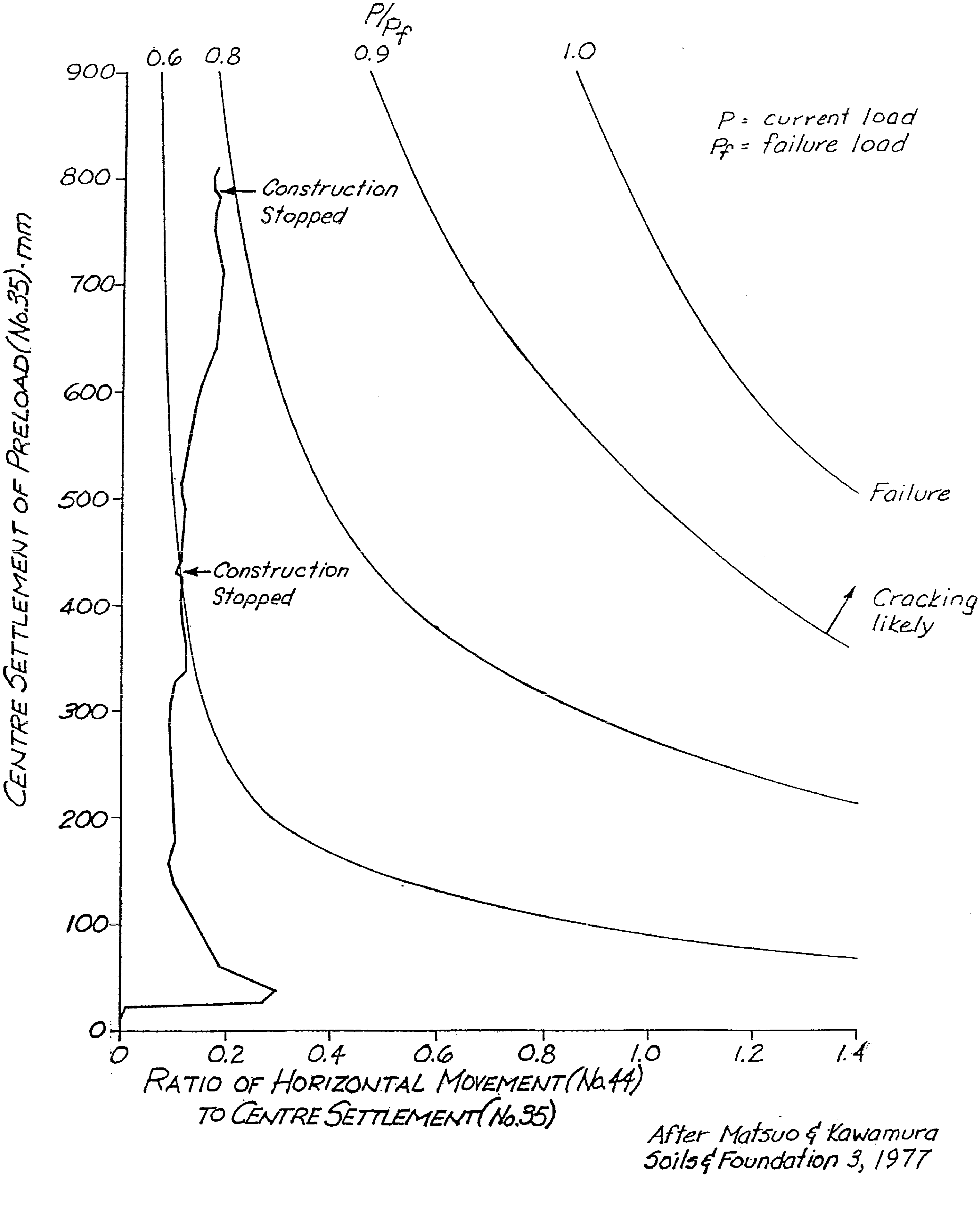

FIGURE 8 STABILITY CHART (DISPLACEMENTS)

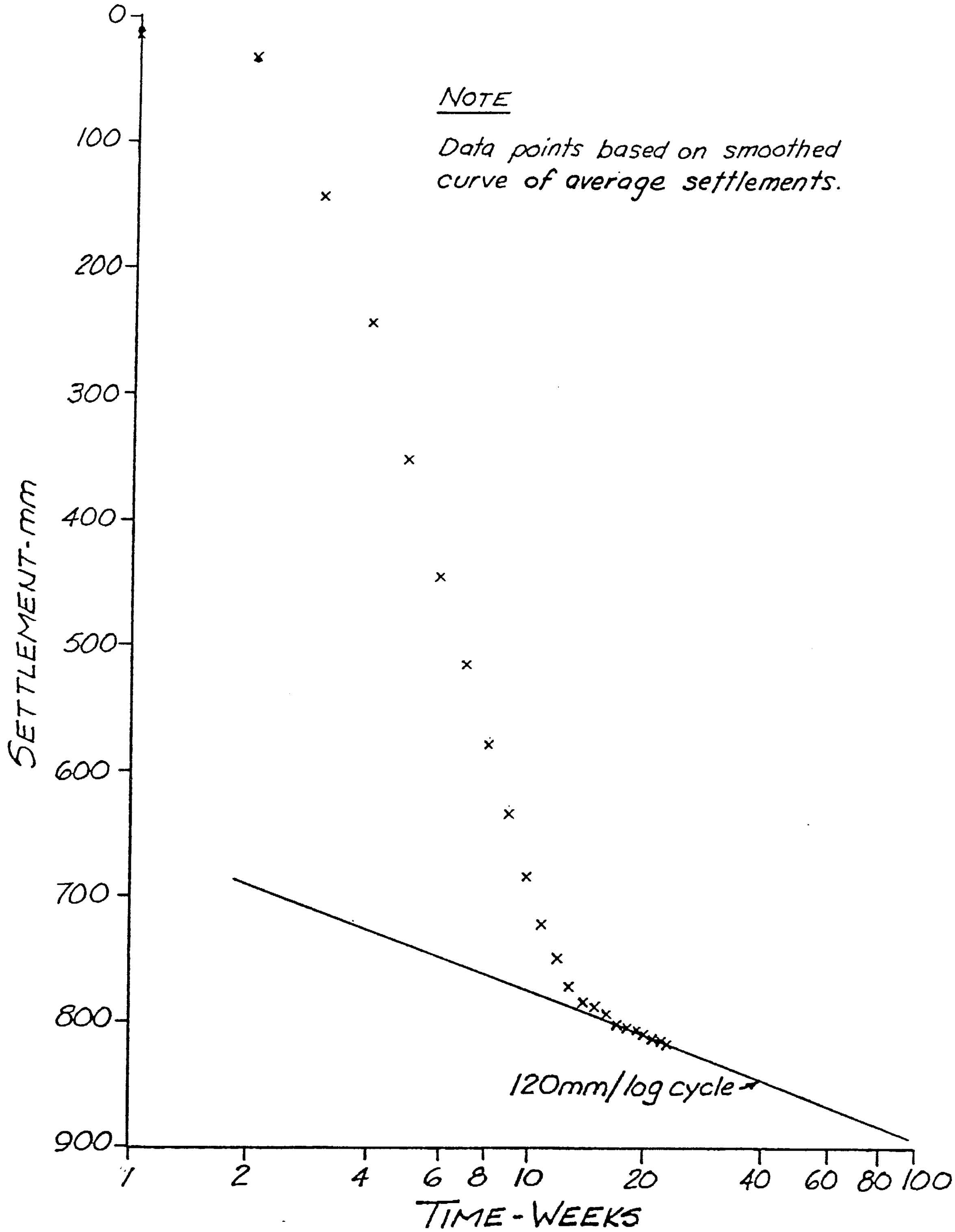

FIGURE 9 PREDICTED LONG TERM SETTLEMENT (LOGARITHMIC)

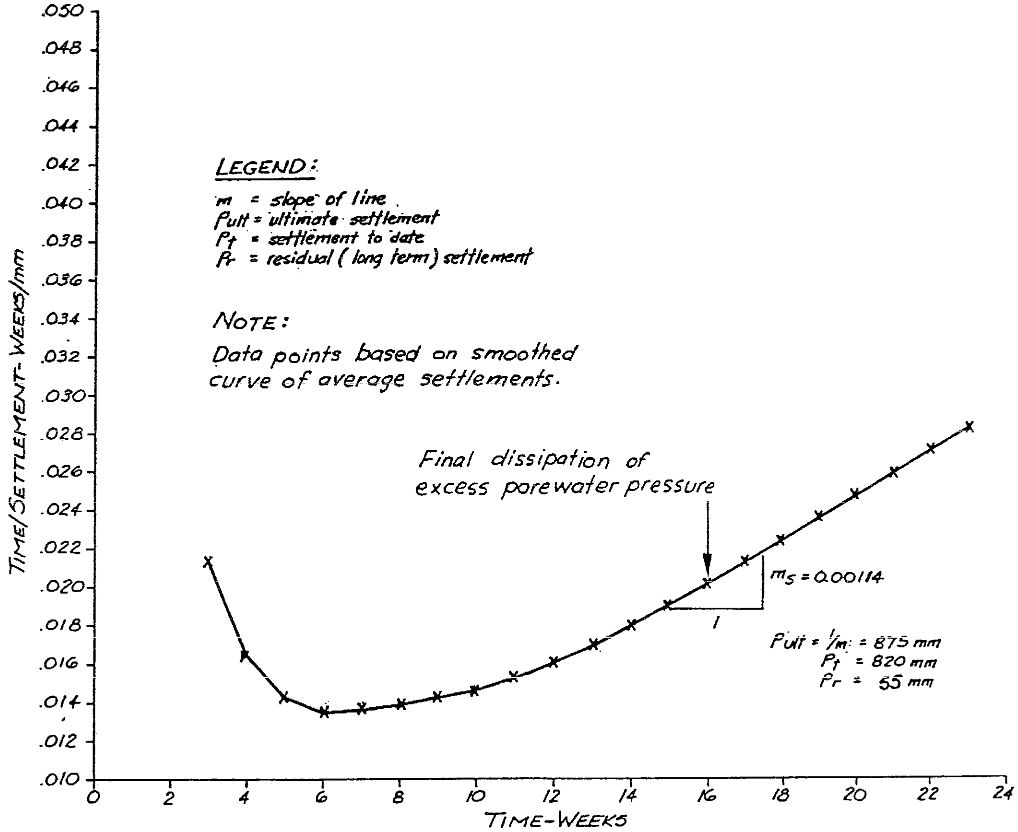

FIGURE 10 PREDICTED LONG TERM SETTLEMENTS (HYPERBOLIC)