Settlements of Bangna-Bangpakong Highway on Soft Bangkok Clay

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Abstract: Time-settlement data of more than 30 different sections of the 55-km long Bangna-Bangpakong Highway in Thailand are analyzed. The deformation parameters, namely: undrained modulus, E_u; drained modulus, E'; and coefficient of consolidation, C_v, are backfigured from the field performance of the highway embankment, and the following correlations are found: $E_u/S_{uv} = 150$; $E'/S_{uv} = 15$; and C_v (field)/C_v (lab) = 26; where S_{uv} is the uncorrected vane shear strength. It was also found that C_v values are overestimated by the method of Asaoka when the during-construction time-settlement curve is used, and are best estimated when backfigured from postconstruction performance. It is also found that for construction settlements, the method proposed by Cox, which is a combination of the method of D'Appolonia et al. for immediate settlements and that of Leroueil et al. for consolidation settlements, underpredicts settlements at some sections but yields conservative estimates when considering secondary settlements since the beginning of construction. Prediction of long-term settlements yields a good estimate for firmer sections using the method of Skempton and Bjerrum, while the method of Asaoka generally underpredicts. The elastic method of Davis and Poulos gives the best estimates of both construction and postconstruction settlements when backfigured parameters are used.

Introduction

Bangna-Bangpakong Highway is a 55-km long major arterial road connecting Bangkok urban area with the eastern part of Thailand (Fig. 1). It was first opened to traffic in 1969, after two years of construction. To cope with the increasing volume of traffic, another carriageway was added parallel to the old one in 1979. During the construction of this latter carriageway, detailed survey was carried out to monitor the performance of the first one. It was found to have suffered severe settlements at several sections. The behavior of this highway provided a unique opportunity to study earth embankments on soft clay since it acted as a full-scale test embankment on subsiding ground conditions. Cox (1981) presented an excellent settlement analysis of this highway. In this paper, the performance of this highway was utilized to verify the applicability of the different methods of settlement prediction, and also to backfigure various soil parameters in actual field conditions that are difficult to determine in normal laboratory tests. Considerable geotechnical data long the highway route are available, together with the time-settlement records at 34 sections. All these sections were analyzed for

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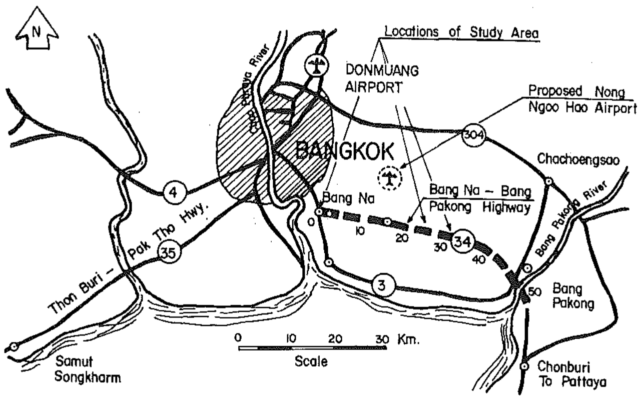

FIG. 1. Location Map (Cox 1981)

construction (drained and undrained), postconstruction, and secondary settlements. Most of the results presented in this paper were based on the work of Ahmed (1987) under the guidance of the first writer.

Geotechnical Characteristics along Bangna-Bangpakong Highway

Past Investigations

Several investigations in the past have been carried out along the alignment of the Bangna-Bangpakong Highway. The first intensive investigation was carried out on the foundation subsoil by the Norwegian Geotechnical Institute (NGI: "Supplementary Geotechnical" 1967) during the design and construction of the highway starting in 1966. Trial embankments were built as described by Eide and Holmberg (1972). Further geotechnical information was obtained during the design and subsequent construction of the second carriageway from 1974 to 1975 and is contained in an unpublished consolidated report by N. D. Lea and Thai Engineering Consultants (N. D. Lea and Associates and Thai Engineering Consultants, unpublished consolidated technical report, 1981). In 1979, during the design of the reconstruction of the old carriageway, several boreholes were drilled and piezometers were installed. Actual settlements were also measured. A summary of this investigation was reported by Cox (1981), as shown in Fig. 2. Thorough investigations of the settlement characteristics of this highway were done by Adhikari (1980), Parnploy (1985), and Pussayanavin and Leerakomsan (1986). A typical embankment cross-section and soil profile at km 30 of the Bangna-Bangpakong Highway are shown in Fig. 3.

Location

The whole stretch of the Bangna-Bangpakong Highway passes over the flat deltaic plain of Thailand, where the subsoil is a soft marine clay called Bangkok clay (Fig. 1). The age of this marine deposit is about 2,000 years, and it is considered as recent deposit (Cox 1981). The thickness of the soft clay varies from 15 m at Bangna (km 0) to 25 m at km 28 from Bangna

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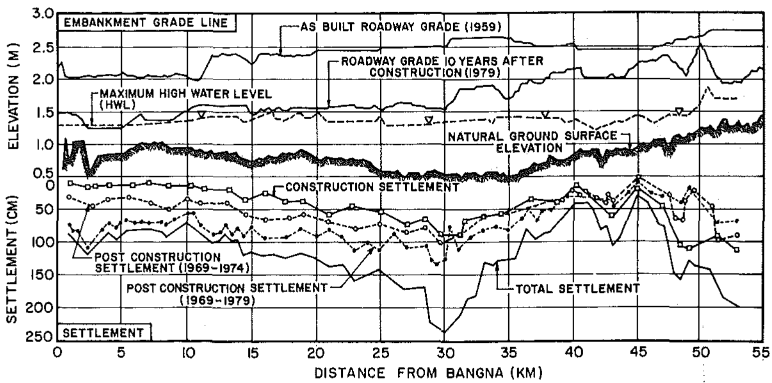

FIG. 2. Ground Elevations and Settlement Profiles (Cox 1981)

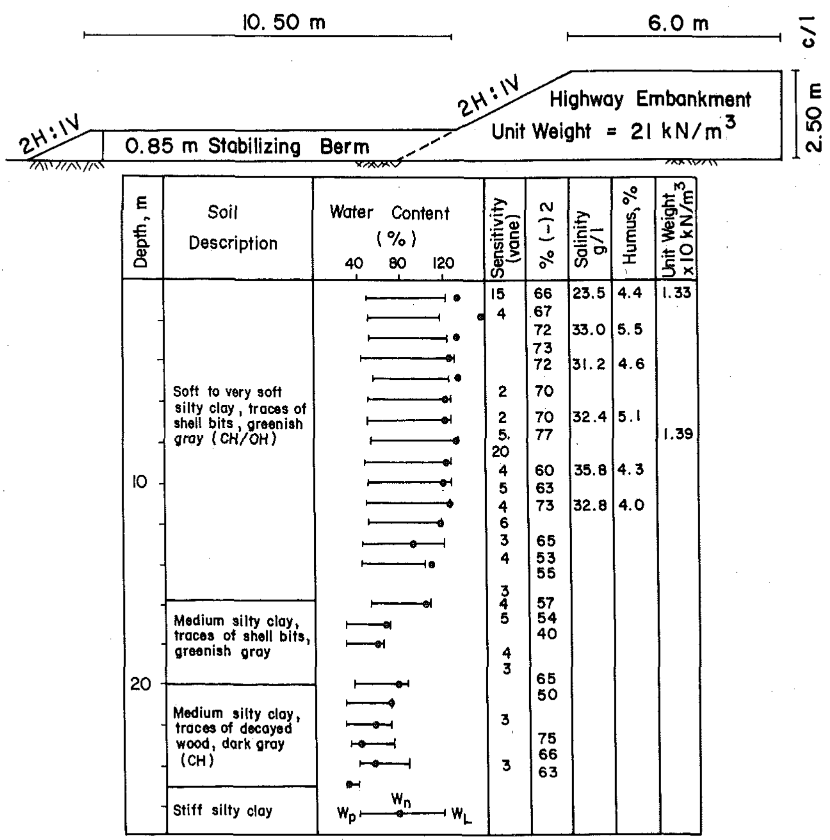

FIG. 3. Typical Embankment Cross-Section and Soil Profile at km 30

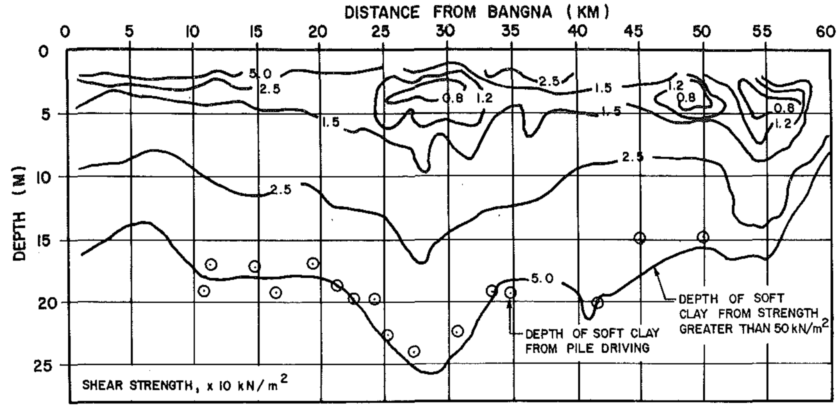

FIG. 4. Undrained Shear Strength Contours Along Highway (Pussayanavin and Leerakomsan (1986)

(Fig. 4). This layer is underlain by stiff clay of 4–10-m thick, followed by a dense to very dense sand. A weathered crust forms the topmost layer that is 1–2-m thick. The natural ground elevation in 1967 varied from 0.8 m above sea level at Bangna, to 0.6 m at km 30–1.2 m near Bangpakong. Ground elevations at Bangna decreased to about 0.60 m in 1979 due to subsidence in this vicinity.

Index Properties

The variation of the natural moisture content along and beneath the highway is given in Fig. 5, where it averages from 80 to 180% in the upper zone, except at km 30 and km 53 where it increased to 140%. The liquidity index profile indicates that the moisture content generally exceeds the liquid limit in the first 5 m of the subsoil, except at km 30 where it remains high.

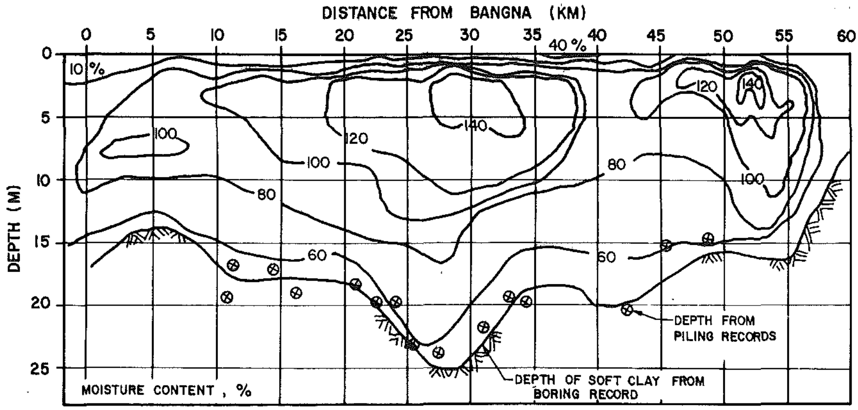

FIG. 5. Moisture Content Profile Along Highway

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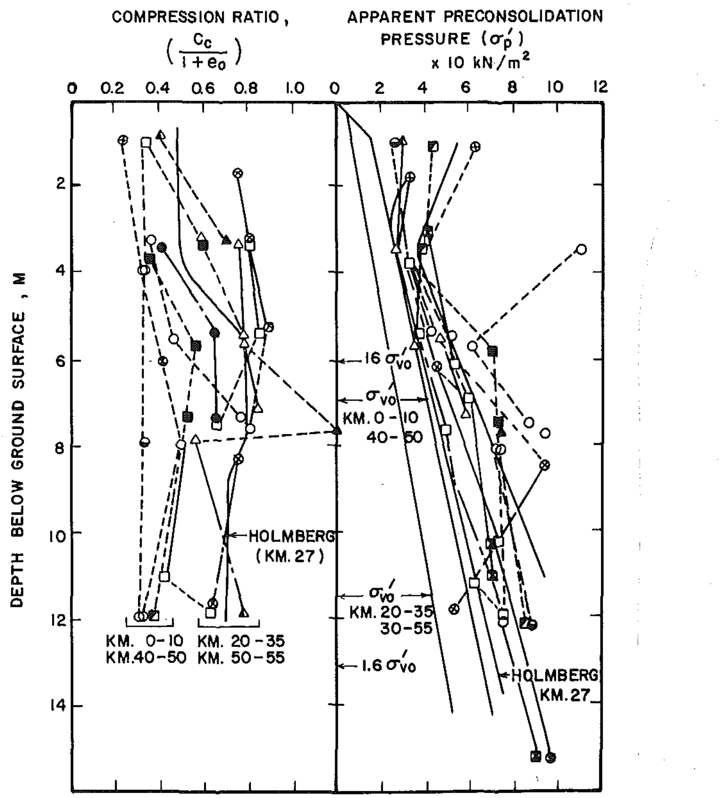

LEGENDS

DISTANCE FROM BANG NA IN KM. (KAMPSAX STATION)

○ 0+842(17+548) N	▲ 28+783(45+013) N	■ 42+776(59+020) N
⊕ 5+389(22+094) A	◭ 28+785(45+013) A	◧ 42+776(59+020) A
● 16+023(32+732) N	□ 33+930(50+160) N	⊗ 53+678(81+735) N
△ 22+717(39+427) N	⊛ 28+779(45+800) T	⊖ 48+971 (76+412) T
	⊠ 16+284 (33+250) T	

σ'_{vo} = AVERAGE OVERBURDEN PRESSURE THROUGHOUT THE PROJECT ROUTE
A = BY ASIAN INSTITUTE OF TECHNOLOGY ——— KM. 0 TO KM. 10, KM. 40 TO KM. 50
N = BY NORWEGIAN GEOTECHNICAL INSTITUTE —— KM. 20 TO KM. 35, KM. 50 TO KM. 55
T = BY THAI ENGINEERING CONSULTANTS

FIG. 6. Compressibility Characteristics at Different Sections Along Highway

This gives an indication of the younger and weaker sediments around this area. The plastic limits are fairly constant with depth, being 30–35% except near softer areas in km 30 where it rises to 40–50%. The liquid limits are high, averaging about 100% in the top 5 m and increasing to 130% in softer areas. The activity of the clay varies from 0.75 to 1.4, with higher values in locations between km 25 and km 50. In km 20–30, the total unit weight averages only 13.5 kN/m^3, whereas in the other sections the average is 14.5 kN/m^3 or higher. The organic content varies from 1 to 8%, while the salt content ranges from 5 to 40 gm/L.

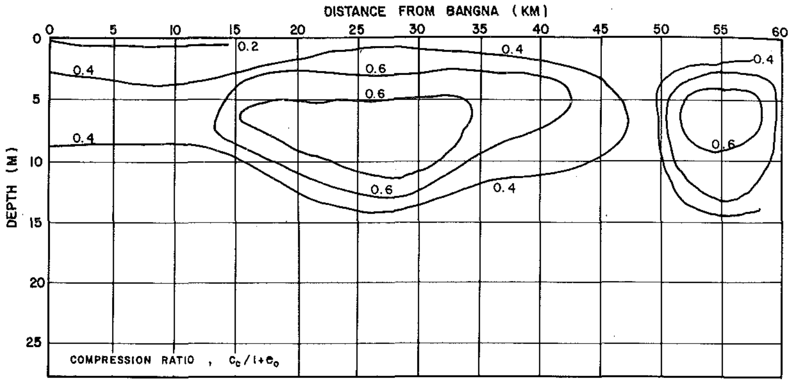

FIG. 7. Compression Ratio Profile Along Highway

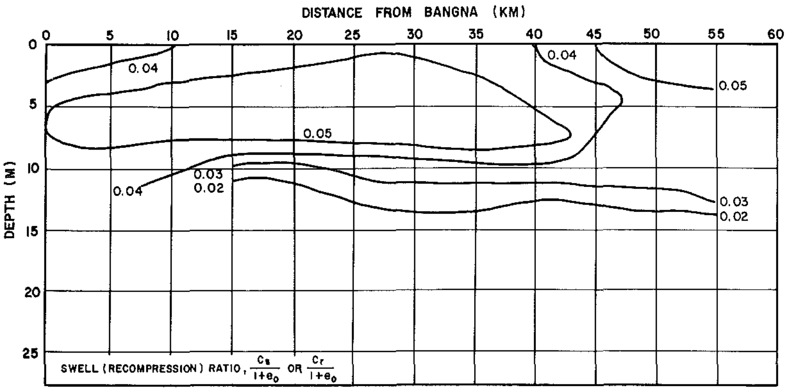

FIG. 8. Recompression Ratio Profile Along Highway

Shear Strength and Compressibility Characteristics

The undrained shear strength, as measured by the field vane, is higher at the topmost weathered crust, reaching minimum value at about 4–5 m, and then increasing with depth (Fig. 4). It can be observed that there are two soft stretches near km 30 and near 50 km. The plots of the compression ratio and apparent maximum past pressure at different sections are shown in Fig. 6, while Figs. 7 and 8 show the contours of the compression and recompression ratios along the highway, respectively. Two main groups of subsoil with different compressibilities are evident from these figures. The first group comprises the soft soils from km 20 to 35 and 50 to 55 km, where the compression ratio averages about 0.80 between 5-m to 10-m depth. The apparent preconsolidation pressure averages 30 kN/m^2 from the surface until 4-m depth, and thereafter increases to 1.6 times the effective overburden pressure. The second group comprises the stronger soils from km 0 to 10 and from km 40 to 50, where the compression ratio varies from 0.30 to 0.50. The apparent preconsolidation pressure varies from 40 to 60 kN/m^2

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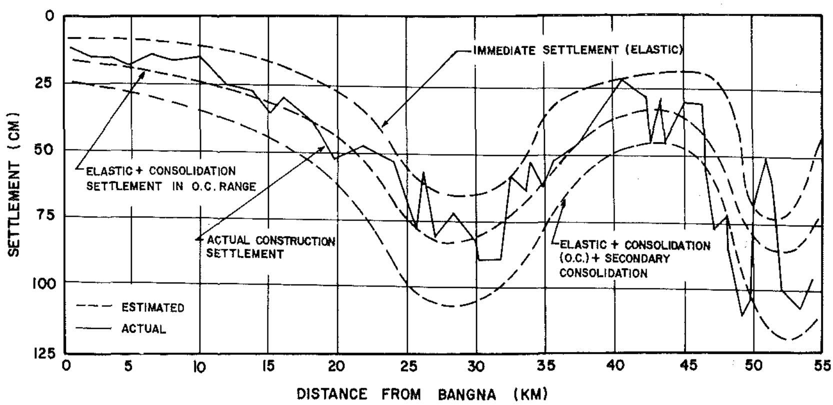

FIG. 9. Comparison of Estimated and Actual Settlements (Cox 1981)

in the surface layers and then increases to 1.6 times the effective overburden pressure. Transitory groups of soils are found between km 10 and 20 and between km 35 and 40, which have compressibilities intermediate between the two main groups.

Elastic Parameters

The undrained elastic modulus, E_u, is commonly expressed as a function of the undrained shear strength, S_u, i.e., $E_u = \alpha \times S_u$. For Bangkok soil, α lies between 70 and 250, where S_u is the uncorrected field-vane strength (Balasubramaniam and Brenner 1981). Trial embankment studies conducted along the nearby Thonburi-Paktho Highway (Cox 1973), reported that E_u = 500 S_u, where S_u is the average undrained shear strength in the weakest zone at any particular section. From the study of the settlement of the adjacent Chachoengsao railway embankment, α was found to be 125 dimensionless (Kampananonda 1984). Results obtained from laboratory stress-path tests (CK_oUC) by Parnploy (1985) at km 2 + 899 of Bangna-Bangpakong Highway, suggested an α value of 253 for weathered clay and an α value of 131 for the soft clay layer. The drained modulus, E', on the other hand, is generally expressed as some fraction of E_u. Kampananonda (1984) reported that E' may be taken as 50% of E_u for the overconsolidated clay at Chachoengsao [overconsolidated ratio (OCR) = 2.5–3.5]. Parnploy (1985) recommended the following correlation between E_u and E': $E' = 0.36E_u$ (weathered clay); $E' = 0.15E_u$ (very soft clay); $E' = 0.26E_u$ (soft clay); and $E' = 0.57E_u$ (medium clay). The undrained Poisson's ratio, ν_u, is usually taken as 0.50. The drained Poisson's ratio, ν', lies between 0.35 to 0.45 for soft clays, and between 0.30 to 0.35 for stiff clays (Poulos 1975). Parnploy (1985) gave a range of 0.30–0.39 for ν', with higher values for normally consolidated soil and lower values for the weathered crust at section 2 + 899 of the Bangna-Bangpakong Highway.

Settlement Characteristics along Bangna-Bangpakong Highway

Actual settlements were measured in 1979 and a summary was presented by Cox (1981), as shown in Fig. 2. For the construction settlements, the

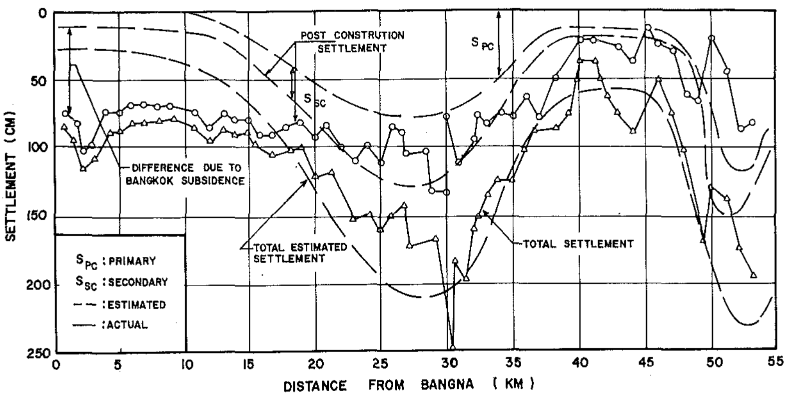

FIG. 10. Comparison of Actual and Estimated Postconstruction Settlements (Cox 1981)

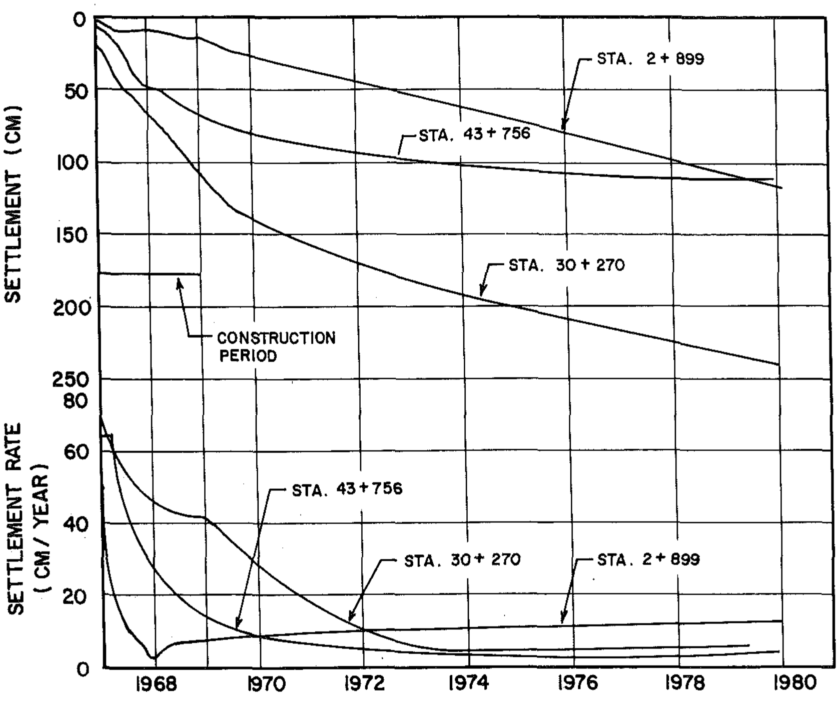

FIG. 11. Typical Time-Settlement Curves for Selected Stations Along the Alignment (Cox 1981)

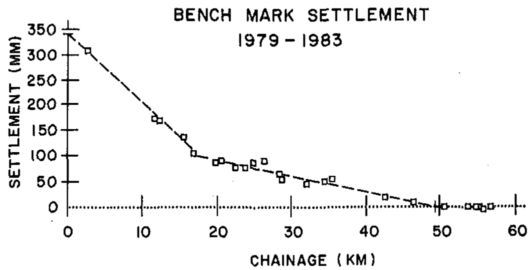

FIG. 12. Settlement Due to Subsidence Along Highway (Pussayanavin and Leerakomsan 1986)

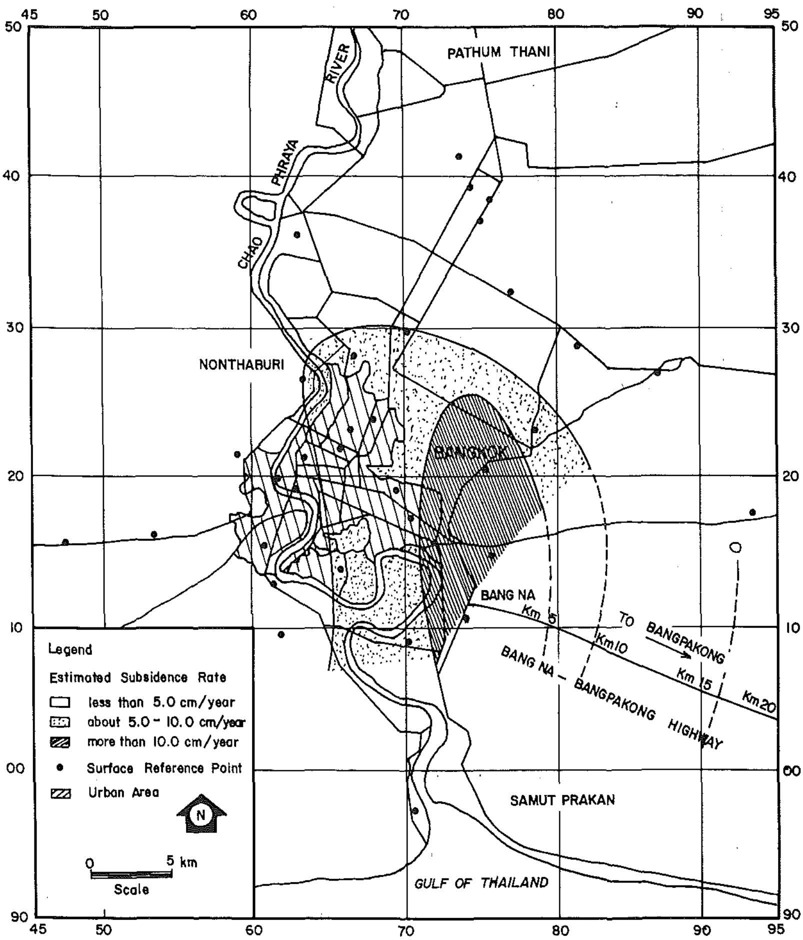

FIG. 13. Zones of Surface Subsidence Rates at the Bangkok Area (AIT: "Investigation" 1982)

actual and predicted settlements are shown in Fig. 9. The prediction was made by combining the method of D'Appolonia et al. (1971) for immediate settlements, and the methods of Leroueil et al. (1978) and Tavenas (1979) for consolidation settlements. The postconstruction settlements, predicted by adding the primary consolidation settlements calculated from the methods of Leroueil et al. (1978) and Tavenas (1979) to the computed secondary settlements, are compared to the actual observed settlements as shown in Fig. 10. Settlement-time relationships for selected stations along the road alignment are shown in Fig. 11. It can be seen from field measurements (Fig. 11) that both the construction and postconstruction settlement rates in the soft areas (e.g., km 30) were high and were typically 40–70 cm/yr. It can also be seen that these settlement rates have slowed down considerably in 1979 and have become typically less than 5 cm/yr. Settlement rates in the stronger sections (between km 40 and 48) have also decreased to very low values of 1–3 cm/yr. However, the settlement rates near Bangna at km 2 + 899 increased to 10 cm/yr instead of decreasing. This was attributed to the effect of Bangkok subsidence [Asian Institute of Technology: "Investigation" (AIT) 1982]. Evidence of subsidence is obtained from the observation that the grade line between km 0 and km 10 have lowered more between 1974 and 1979 than it did between 1967 and 1979. The rate of settlement near km 0 in 1979 is similar to the subsidence around Bangna (Fig. 12). Fig. 13 shows the zones of surface subsidence rates in Bangkok for the period of 1978–1979. The correlation between subsidence settlement and the piezometric level drop has been established and reports by AIT (AIT: "Investigation" 1982) indicated that Bangna area is one of the worst subsiding areas of Bangkok and is subsiding at a rate of about 10–15 cm/yr. It was found that the compression of the soft clay layer occurs due to the drawdown of piezometric pressures caused by the ground-water withdrawal. Further evidence of subsidence is obtained from the difference of natural ground elevations in 1966 and 1976. The settlement rates between 1974 and 1979 increased continuously as one proceeds from Bangpakong to Bangna, in spite of the fact that the soil condition was stronger towards Bangna. Total settlement due to subsidence at Bangna from 1969 to 1979 is estimated to be about 40 cm (N.D. Lea and Associates and Thai Engineering Consultants Co., unpublished consolidated technical report, 1981).

METHODS OF SETTLEMENT PREDICTION

Immediate Elastic Settlement

Elastic undrained settlement of a loaded area of width B is given by the following formula (Simons and Menzies 1977):

$$\rho_e = \frac{qBI}{E_u} \qquad (1)$$

where q is the applied load per unit area, B is the width of the loaded area, E_u is the undrained modulus, and I is the influence factor. This formula is applicable for a fairly homogeneous soil stratum where it is possible to use only one average value for E_u. However, the variation of E_u with depth can also be taken into account by replacing the multilayered system with one hypothetical layer on a rigid base, such that the depth of this hypothetical layer is successively extended to incorporate each real subsoil layer. Design charts, such as those given by Janbu et al. (1956), are available to estimate the undrained settlements.

D'Appolonia et al. (1971) developed a simplified method to account for local yielding beneath the embankment as it approaches failure. This local yielding occurs when the shear stresses at some points of the soil mass are relatively high compared to the shear strength during the undrained loading. The effect becomes eminent when the applied stress exceeds 20% of the ultimate bearing stress for normally consolidated soils to as high as 60% for overconsolidated soils. The immediate elastic settlement, including the effect of local yielding, can then be estimated from the following formula:

$$p_i = \frac{qBI(1 - \nu_u^2)}{E_u \times SR} \qquad (2)$$

where SR is the settlement ratio that is a function of the initial shear-stress ratio (f), the applied stress ratio (q/q_{ult}), and the geometry of the problem.

Construction Settlement

Leroueil et al. (1978) and Tavenas (1979) suggested that the consolidation settlement in the overconsolidated range should also be taken into account in calculating construction settlements. Due to a high value of C_v for an overconsolidated soil, the dissipation of excess pore pressure will take place quickly, resulting in an increase in the effective stress. This process will continue until the effective stress reaches the maximum past pressure, after which C_v will drop sharply. Thus, consolidation settlement during construction, p_{cc}, can be estimated by:

$$p_{cc} = \frac{C_r}{1 + e_0} H \log \frac{\sigma'_{v0} + \Delta\sigma'_v}{\sigma'_{v0}}; \qquad (\sigma'_{v0} + \Delta\sigma'_v) \le \sigma'_p \qquad (3)$$

where C_r is the recompression index of the soil, H is the thickness of compressible layer, σ'_{v0} is the in situ vertical effective stress and $\Delta\sigma'_v$ is the increase in vertical stress due to the loading. Cox (1981) adopted the combination of the aforementioned method and the method of D'Appolonia et al. (1971) to estimate the construction settlement of the Bangna-Bangpakong Highway as shown in Fig. 9.

Consolidation Settlement

Skempton and Bjerrum (1957) developed a method in which the final primary consolidation settlement is calculated from three-dimensional excess pore pressure obtained under undrained axisymmetric (triaxial) stress conditions and can be written in terms of the consolidation settlements computed from oedometer tests as:

$$p_c = \mu p_{\mathrm{oed}} \qquad (4)$$

The correction factor μ is a function of the pore-pressure coefficient A and the geometry of the problem. For Bangkok soil, Balasubramaniam et al. (1985) obtained a chart showing the correlation of μ with OCR. In the chart, it is indicated that $\mu = 1$ for a normally consolidated soil, and decreases with the increase in OCR, taking a constant value of 0.4 when OCR = 3 or more.

Davis and Poulos (1968) suggested the use of elastic theory to compute the total final settlement of a layered soil, which can be obtained by the summation of the vertical strain in each layer. The long term drained settlement (p_d) can be computed as:

$$p_d = \Sigma \frac{H}{E'} [\Delta\sigma_z - \nu'(\Delta\sigma_x + \Delta\sigma_y)] \quad (5)$$

where E' and ν' are the drained elastic parameters for the soil structure appropriate to stress changes in each layer.

An observational procedure of settlement prediction was proposed by Asaoka (1978), based on the one-dimensional consolidation equation of Mikasa (1965), in which the governing differential equation for settlement-time relationship is approximated by an autoregressive model of finite order, such that for the case of a first-order autoregressive model, and a load that is constant with time, it is possible to solve the final consolidation settlement (p_f) and the coefficient of consolidation, C_v, based on the settlement-time history of the soil. The graphical method proposed by Asaoka (1978), as one possible way to solve the first-order autoregressive model, consists of the following steps:

1. The observed time-settlement curve plotted to an arithmetic scale, is divided into equal time intervals, Δt, which usually ranges from 30 to 100 days. The settlements, p_1, p_2, ... corresponding to times t_1, t_2, ... are read off and tabulated.
2. The settlement values p_1, p_2, ... are plotted as points (p_{i-1}, p_i) in a coordinate system with axis p_{i-1} and p_i. A 45° line is then drawn.
3. The plotted points are then fitted by a straight line whose corresponding slope is read as β_1. The point of intersection with the 45° line, gives the final consolidation settlement, p_f. The coefficient of consolidation can then be calculated from

$$C_v = -\frac{5}{12} H^2 \frac{\ln \beta_1}{\Delta t} \quad (6)$$

For the estimation of ultimate consolidation settlement, Cox (1981) used the expression suggested by Leroueil et al. (1978) and Tavenas (1979) while taking into consideration the effect of secondary settlement as follows:

$$p_c = \frac{H}{1 + e_0} \left(C_c \log \frac{\sigma'_{v0} + \Delta\sigma'_v}{\sigma'_p} + C_\alpha \log \frac{t_1}{t_2} \right) \quad (7)$$

where C_c and C_α are the compression index and the coefficient of secondary consolidation of the soil, respectively. A reasonable agreement between actual settlement measured in 1979 and the estimated ultimate settlement from the aforementioned formula has been reached except in the first 20 km from Bangna, where the effect of Bangkok subsidence is very high (Cox 1981).

Secondary Consolidation Settlement

Since secondary consolidation settlement occurs after an extended period of time and is seldom observed, there is little possibility of checking the validity of secondary compression. In practice, Raymond and Whals (1976) proposed the following equation to compute the amount of secondary compression:

$$p_{sec} = \frac{C_\alpha}{1 + e_0} H \log \frac{t_p}{t_s} \quad \text{(8)}$$

where t_s is the time from the instant when secondary consolidation is assumed to commence and t_p is the time at which secondary compression is desired. For the analysis of Bangna-Bangpakong Highway, C_α/C_c was taken as 0.045 as suggested by Cox (1981).

Method of Analysis

According to the classification of Gioda (1985), the method of back analysis followed in this study may be termed as a combination of the inverse and direct approaches. The soil parameters were first obtained by the inverse approach, and then further modified by the direct approach as detailed in the following steps:

1. The coefficient of consolidation, C_v, was obtained by Asaoka's method using a generalized computer program. Actual construction settlement was read off from the field time-settlement curves. The time when 100% consolidation settlement is over was estimated from the value of C_v previously obtained using the curves of Olson (1977). Secondary settlement was then estimated using Eq. 8 and taken out from the corresponding field measurement. The effect of subsidence was also subtracted in the zone of km 0–km 18. The remaining part (p_d) contains immediate (p_i) and consolidation (p_c) settlements only. The degree of consolidation just after construction (U_c) is then estimated from the values of C_v previously obtained.
2. Immediate settlement was obtained from Eq. 2. The undrained shear strength, S_u, was read off from Fig. 4 for each layer to estimate E_u that was expressed as αS_u. The subsoil was then divided into three layers, namely: from 0- to 5-m depth, 5- to 10-m depth, and 10-m depth down to the top of the stiff clay. The value of *SR* was taken from the design parameters recommended by N. D. Lea and Thai Engineering Consultants (N. D. Lea and associates and Thai Engineering Consultants, unpublished consolidated technical report, 1981). The consolidation settlement during construction (from 1967 to 1969) was then estimated as $p_{cc} = (p_d - p_i) \times (U_c)$, where p_d, U_c, and p_i are as previously obtained. The actual construction settlement (p_{cons}) was read off from the field-time settlement curves. The value of α is then evaluated from $p_{cons} = p_i + (p_d - p_i) \times (U_c)$. The drained settlement was estimated from Eq. 5 in terms of β ($\beta = E'/S_u$), where β is then evaluated by equating with p_d from step 1.
3. Representative values of α and β were chosen to carry out settlement analysis at selected sections (kms 5, 10, 15, ..., 45, 50, and 53). The immediate and drained settlements were estimated, and U_c was evaluated from the knowledge of actual construction settlements. The corresponding value of T_c was obtained from the relationships proposed by Olson (1977), and C_v was then recalculated from $C_v = T_c H^2/t_c$. Settlement values at 1, 2, 5, and 10 years after construction were then estimated as $p_{total} = p_i + (p_d - p_i) \times (U) + p_{sec} + p_{sub}$, where p_{sub} is the subsidence settlement at the time of interest (where applicable).

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Secondary settlements were assumed to be small, and were neglected, at the sections where the maximum past pressure was not exceeded by the embankment loading. The end-of-primary consolidation settlement was then estimated at the time when the degree of consolidation was 90% or higher, as evaluated from recalculated C_v values. Then, the corresponding values of subsidence were added.

Interpretation of Results

Undrained Modulus

The normalized value of E_u with respect to S_u for the whole stretch of the Bangna-Bangpakong highway was obtained. It was observed that the value of α lies between 100 and 250 along the highway. The range obtained for α agreed well with the existing literature. For soft Bangkok clay, α varies between 70 and 250, where S_u is the field-vane strength (Balasubramaniam et al. 1981). Finite element analysis of settlement of the nearby Nong Ngoo Hao clay, reported an α of 70, which gives the best agreement with the measured values (Sivandran 1985). Parnploy (1985) suggested α values of 253 for the weathered clay, and 131 for soft clay at km 2 + 899 as determined from CK_oUC stress-path tests. Bergado et al. (1986) correlated the E_u value obtained from pressuremeter test to the vane shear strength of the soil and obtained an α of 145. The mean and standard deviations of α obtained from this study were 168 and 67.3, respectively. From a histogram drawn for α, a representative value of 150 is recommended.

Drained Modulus

The ratio of the drained modulus, E', and undrained shear strength, S_u, estimated for the 26 sections of the Bangna-Bangpakong highway were computed and it was found that most of the points lie within the band between 15 and 40. The mean and standard deviations are 35 and 21.4, respectively. Further analysis of long-term drained settlements, yielded β of 15 for the best estimation of field conditions. The value is somewhat lower than that suggested by Parnploy (1985), who reported a value of 20 for the soft clay and about 144 for the weathered clay. This difference may be due to the inclusion of the effects of both immediate elastic and inelastic deformations in calculating E'. The E'/E_u ratio was 0.10.

Coefficient of Consolidation

The field values of coefficient of consolidation, C_v, estimated from the during-construcion time-settlement behavior of the embankment using Asaoka's (1978) method, were slightly higher as compared to the corresponding values estimated by Parnploy (1985) at km 2 + 899 of the main embankment. This disagreement may be due to the use of different parts of the time-settlement curve although both values appeared to be rather high, which was also confirmed later in the estimation of the construction settlement. The higher values obtained for C_v, in this study, may be due to the following reasons:

1. The factor of safety against failure is very low in most parts of the highway (FS = 1.3), such that large creep settlements can be expected, which were assumed to commence just after construction. Thus, C_v value is further lowered as the settlement goes on, and may be overestimated, when calculated based on the initial time-settlement curve.

TABLE 1. Field and Laboratory Values of C_v and Their Ratios at Different Sections of the Highway

Station (km) (1)	C_v (lab) (m^3/yr) (2)	C_v (field) (m^2/yr) (3)	Ratio (4)	Source of C_v (lab) (5)
2 + 899	0.34	12	35	Parnploy (1985)
21 + 290	1.46	15	10	Adhikari (1980)
27 + 000	0.63	26	41	Eide and Holmberg (1984)
27 + 170	1.28	26	20	Adhikari (1980)

2. Rapid consolidation close to the drainage boundaries, partly due to the effect of ground subsidence, leads to the formation of multidimensional drainage.

Parnploy (1985) estimated creep strain at 15, 65, and 20% of undrained deformation in the layers from 1.8- to 4.0-m, 4.0- to 7.0-m, and 7.0- to 9.0-m depths, respectively. A new set of C_v was obtained at selected sections (kms 0, 10, ..., 50, and 53) by analyzing their corresponding construction settlement. The resulting value of C_v was seen to vary between 12 and 28 m^2/yr along the highway. It was also estimated that about 85% consolidation settlement had taken place until 1979, which agreed well with that reported by N. D. Lea and Thai Engineering Consultants (N. D. Lea and associates and Thai Engineering Consultants, unpublished consolidated technical report, 1981) for the degree of consolidation (relative to pore pressure changes) of about 50–100%, with an average of about 70%. Laboratory values of C_v obtained from the standard consolidation test at four sections, namely: station 2 + 899 (Parnploy 1985), 21 + 290 (Adhikari 1980), 27 + 000 (Eide and Holmberg 1984), and 27 + 170 (Adhikari 1980), were compared to the field values as shown in Table 1. The ratio C_v(field)/C_v(lab) was seen to vary between 10 and 41, with mean and standard deviations of 26.5 and 14, respectively.

This high ratio of C_v(field)/C_v(lab) has also been found by previous investigators. Kampananonda (1983) recommended a ratio of 70 while Balasubramaniam et al. (1985) obtained ratios ranging from 2 to 200. The high ratio of consolidation coefficients may be caused by the presence of numerous fine sand lenses and silt seams (Bergado et al. 1987); by the presence of fissures and rootholes (Moh et al. 1972); and by the multidimensional drainage paths created by the effects of ground-water pumping from the underlying aquifers that caused ground subsidence (Bergado et al. 1988). The C_v(lab) values may be underestimated in the normal oedometer test, wherein relatively small and "homogeneous" specimens are usually selected for testing. The C_v(field) values may also be overestimated when backcalculated by the method of Asaoka (1978) that is based on actual settlement curve that may include creep settlements, although adjustments have been made by a direct method as stated previously.

Settlement Analysis

Settlements at different intervals of time along the whole stretch of the highway was estimated using the parameters obtained. Known values of subsidence were added to these estimated settlements from km 0 to 18, as it

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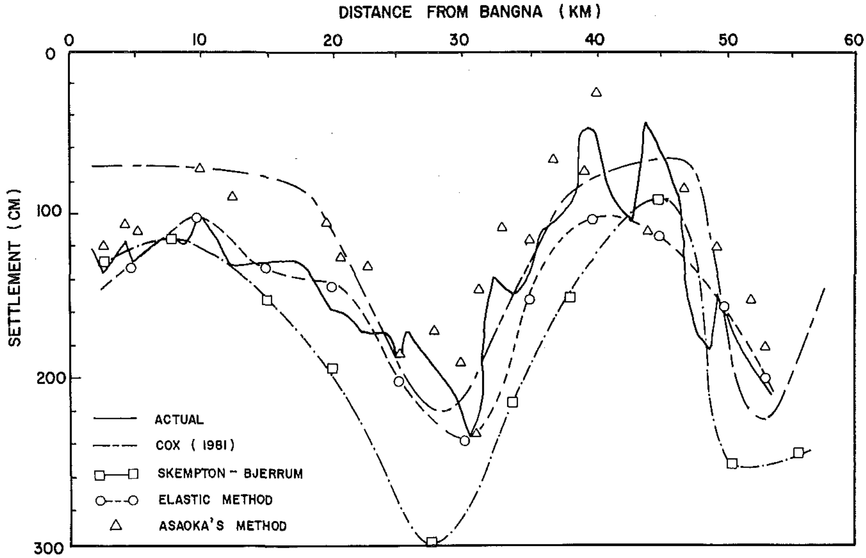

FIG. 14. Calculated End-of-Primary Consolidation Settlements Using Different Methods

was seen that in 1979 the influence of Bangkok subsidence extended up to km 18. Secondary settlement also contributed significantly in areas where the maximum past pressure was exceeded due to the embankment loading. It was found that there are two zones, one between km 0 and 15, and the other between km 40 and 45, where the stresses generated by the embankment load did not exceed the maximum past pressure and thus, secondary settlement was neglected. In the remaining areas (i.e., km 15–40 and km 45–53), secondary settlement was considered and was assumed to commence just after construction.

The long-term settlements estimated from different methods are compared to the actual measured settlement in 1979 as shown in Fig. 14. The initial, secondary, and subsidence settlements were also accounted for where applicable. It can be seen that a close agreement between the estimated values and the measured values resulted using the elastic method. Final consolidation settlements estimated by the Skempton-Bjerrum (1957) method agreed with the elastic approach in firmer areas. This may be due to the method's inherent weak point, that it assumes the distribution of total stresses imposed by the foundation load to be unchanged during consolidation process, regardless of the change in Poisson's ratio from its undrained to drained value (Balasubramaniam et al. 1981). Asaoka's (1978) method generally underpredicted the settlements along the highway. This may be because the initial behavior of the soil reflected the nature of an overconsolidated soil of low compressibility, as pore water only from the overconsolidated zone dissipated during construction. The total settlements estimated by Cox (1981) is also superimposed in Fig. 14 and it can be seen that the method slightly underpredicted the settlements at firmer sections.

Conclusions

From the back-analysis procedure followed in this study, the following conclusions can be drawn:

1. The field values of the coefficient of consolidation, C_v, varies from 12 to 28 m^2/yr and are representative of the present stress level. The ratio between the field and laboratory values of C_v was found to be about 26.
2. The settlement of the Bangna-Bangpakong Highway was best estimated by taking $E_u = 150S_{uv}$. For the value of the drained modulus to estimate long-term settlements, $E' = 15S_{uv}$ was found to be the most suitable. Thus, E'/E_u was found to be 0.10.
3. Skempton-Bjerrum (1957) method gave a good estimation of the long-term settlements in the stronger sections.
4. Asaoka's (1978) method underpredicted the total observed construction settlement along the whole stretch of the highway.
5. A conservative estimate of construction settlement can be obtained by combining the methods of D'Appolonia et al. (1971) for immediate settlement and the method of Leroueil et al. (1978) for consolidation settlement during construction, then adding the corresponding secondary settlement.
6. The elastic method of settlement prediction was found to give a good estimation of both construction and postconstruction settlements when backfigured soil parameters were used.

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Appendix I. References

Adhikari, A. K. (1980). "Settlement characteristics of Bangna-Bangpakong Highway," thesis presented to the Asian Institute of Technology at Bangkok, Thailand, in partial fulfillment of the requirements for the degree of Master of Engineering.
Ahmed, S. (1987). "Predicted and observed settlement characteristics of Bangna-Bangpakong Highway," thesis presented to the Asian Institute of Technology at Bangkok, Thailand, in partial fulfillment of the requirements for the degree of Master of Engineering.
Asaoka, A. (1978). "Observational procedure for settlement prediction." *Soils and Foundations,* 18(4), 87–101.
Balasubramaniam, A. S., and Brenner, R. P. (1981). "Consolidation and settlement of soft clay." *Soft Clay Engineering,* Brand, E. W. and R. P. Brenner, eds., Elsevier Scientific Publishing Co.
Balasubramaniam, A. S., et al. (1985). "Stability and settlement characteristics of structures in soft Bangkok clay." *Proc. 11th Int. Conference on Soil Mech. and Foundation Engrg.,* International Society of Soil Mechanics and Foundation Engineering, 3, 1641–1648.
Bergado, D. T., et al. (1986). "Correlations of in situ tests in Bangkok subsoils." *Geotech. Engrg.,* 17(1), 1–36.

Bergado, D. T., et al. (1987). "Reliability assessment of test embankments on soft Bangkok clay by variance reduction and nearest-neighbor methods." *Computers and Geotech.*, 4, 171–194.
Bergado, D. T., et al. (1988). "Causes, effects, and predictions of land subsidence in AIT campus, Chao Phraya Plain, Thailand." *Bulletin Assoc. Engrg. Geol.*, 25(1), 57–81.
Cox, J. B. (1973). "Trial embankment studies of Thonburi-Paktho Highway, Thailand." *LEA-GECO Int. Tech. Rep. R-5*, Bangkok, Thailand.
Cox, J. B. (1981). "The settlement of a 55-km long highway on soft Bangkok clay." *Proc. 10th Int. Conference on Soil Mech. and Foundation Engrg.*, 1, 101–104.
D'Appolonia, D. J., Poulos, H. G., and Ladd, C. C. (1971). "Initial settlement of structures on clay." *J. Soil Mech. and Found. Engrg. Div.*, ASCE, 97, 1359–1377.
Davis, E. H., and Poulos, H. G. (1968). "The use of elastic theory for settlement prediction under three-dimensional conditions." *Geotechnique*, London, England, 18, 67–91.
Eide, O., and Holmberg, S. (1972). "Test fills to failure on soft Bangkok clay." *Proc. ASCE Spec. Conf. on Perf. of Earth and Earth-Supported Struc.*, Purdue University, 1, 159–180.
Gioda, G. (1985). "Some remarks on back-analysis and characterization problems." *Proc. 5th Intl. Conf. Numerical Methods Geomech.*, Nagoya, 1, 47–61.
"Investigation of land subsidence caused by deep well pumping in the Bangkok area." (1982). *Final Asian Institute of Technology Research Report No. 144*, National Envir. Board, Bangkok, Thailand.
Janbu, N., Bjerrum, L., and Kjaernsli, B. (1956). "Veilidning ved Losning av Fundamenteringsoppgaver." *Norwegian Geotech. Inst. Pub.*, 16 (in Swedish).
Kampananonda, N. (1984). "Settlement prediction and performance of railway embankment at Chachoengsao," thesis presented to the Asian Institute of Technology at Bangkok, Thailand, in partial fulfillment of the requirements for the degree of Master of Engineering.
Leroueil, S., et al. (1978). "Construction pore pressures in clay foundations under embankments, Part II: generalized behaviour." *Canadian Geotech. J.*, 15(1), 66–82.
Mikasa, M. (1965). "The consolidation of soft clay." *A new consolidation theory and its applications*, Kajima Publishing Co., Tokyo, Japan, (in Japanese).
Moh, Z. C., Brand, E. W., and Nelson, J. D. (1972). "Pore pressure under a bund on soft fissured clay." *Proc. performance of earth and earth-supported structures*, Purdue Univ., 1, 243–272.
Olson, R. E. (1977). "Consolidation under time-dependent loading." *J. Geotech. Eng. Div.*, ASCE, 103(1), 55–60.
Parnploy, U. (1985). "Deformation analysis and settlement prediction of Bangna-Bangpakong Highway (section 1)," thesis presented to the Asian Institute of Technology at Bangkok, Thailand, in partial fulfillment of the requirements for the degree of Master of Engineering.
Poulos, H. G. (1975). "Settlement of isolated foundations." *Proc. Symp. on Soil Mechanics-Recent Developments*, University of New South Wales, Australia, 181–212.
Pussayanavin, P., and Leerakomsan, S. (1986). "Design and construction of Bangna-Bangpakong highway." *Proc. Southeast Asian Congress on Roads, Highways, and Bridges*, Kuala Lumpur, Malaysia, H1–H28.
Raymond, G. P., and Whals, H. E. (1976). "Estimating one-dimensional consolidation including secondary compression on soft clay loaded from overconsolidated to normally consolidated state." *Spec. Rep. 164*, Transp. Res. Board, 17–23.
Simons, N. E., and Menzies, B. K. (1977). *A short course in foundation engineering*, Butterworth and Co., Ltd., London, England, 37.
Sivandran, C. (1975). "Finite element analysis of test embankment and excavation at Nong Ngoo Hao," thesis presented to the Asian Institute of Technology at Bangkok, Thailand, in partial fulfillment of the requirements for the degree of Master of Engineering.

Skempton, A. W., and Bjerrum, L. (1957). "A contribution to the settlement analysis of foundations on clay." Geotechnique, London, England, 7, 168–178.
"Supplementary geotechnical investigations, Bangkok-Siracha Highway." (1967). *Report 66/36, 1-10,* Norwegian Geotech. Inst., Dept.of Highway of Thailand.
Tavenas, F. (1979). "The behaviour of embankments on clay foundations: state-of-the-art." *Proc. 32nd Canadian Geotech. Conf.,* Canadian Geotechnical Society.

Appendix II. Notation

The following symbols are used in this paper:

B = width of loaded area;
C_c = compression index of soil;
C_r = recompression index of soil;
C_v = coefficient of consolidation;
C_α = coefficient of secondary consolidation;
E_u = undrained modulus;
E' = drained modulus;
e_o = initial void ratio;
f = initial shear stress ratio;
H = layer thickness;
I = influence factor;
p_c = primary consolidation settlement;
p_{cc} = consolidation settlement during construction;
p_{cons} = actual construction settlement;
p_d = long term drained settlement;
p_e = elastic undrained settlement;
p_i = immediate settlement;
p_{oed} = consolidation settlement from oedometer test;
p_{sec} = secondary consolidation settlement;
p_{sub} = subsidence settlement;
p_{total} = total settlements;
q = applied load per unit area;
q_{ult} = ultimate load per unit area;
SR = settlement ratio;
S_u = undrained shear strength;
S_{uv} = undrained vane shear strength;
T_c = dimensionless time factor;
t = time;
t = time increment;
t_c = consolidation time;
t_p = time where secondary compression is desired;
t_s = time when secondary consolidation is assumed to commence;
U = degree of consolidation;
U_c = degree of consolidation just after construction;
α = undrained parameter (E_u/S_u);
β = drained parameter (E'/S_u);
β_1 = slope of time-settlement records;
σ'_p = maximum past pressure;
σ'_{vo} = in situ vertical stress;
$\Delta\sigma'_v$ = increase in vertical stress due to loading;

$\Delta\sigma_x$ = increase in vertical stress along x-direction;
$\Delta\sigma_y$ = increase in vertical stress along y-direction;
$\Delta\sigma_z$ = increase in vertical stress along z-direction;
μ = pore pressure parameter;
ν_u = undrained Poisson's ratio; and
ν' = drained Poisson's ratio.