Inverse analysis of settlement characteristics of Bangna-Bangpakong Highway, Thailand in subsiding environment

L'analyse inverse des caractéristiques de tassement de la route Bangna-Bangpakong, Thailande

D.T.BERGADO, GTE Division, Asian Institute of Technology, Bangkok, Thailand
S.AHMED, GTE Division, Asian Institute of Technology, Bangkok, Thailand
C.L.SAMPACO, GTE Division, Asian Institute of Technology, Bangkok, Thailand
B.N.LEKHAK, GTE Division, Asian Institute of Technology, Bangkok, Thailand

SYNOPSIS: Time-settlement data of more than 30 different sections of the 55 km long Bangna-Bangpakong Highway were studied by inverse analysis. The deformation parameters, namely: undrained modulus, E_u; drained modulus, E'; and coefficient of consolidation, C_v, were back-figured from the field performance of the highway embankment and the following correlations were found: $E_u/S_{uv} = 150$, $E'/S_{uv} = 15$, and $C_v(\text{field})/C_v(\text{lab}) = 26$, where S_{uv} is the vane shear strength. It was also found that C_v values were overestimated by the method of Asaoka (1978) when the during-construction time-settlement curve was used, and best estimated from post-construction data. For prediction of construction settlements, the method of Cox (1981) which is a combination of the method of D'Appolonia et al (1971) for immediate settlements and that of Leroueil et al (1978) for consolidation settlements, underpredicted settlements at some sections but yielded conservative estimates by adding secondary settlements since the beginning of construction. A good estimate of long term settlement was obtained for firmer sections by the method of Skempton & Bjerrum (1957) and the method of Asaoka (1978) generally underpredicted. The elastic method of Davis & Poulos (1968) gave the best estimates of both construction and post construction settlements when back-figured parameters were used.

1 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 open 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. The performance of this highway was utilized to verify the applicability of the different methods of settlement prediction as well as to back-analyze (inverse analysis) various soil parameters in actual field conditions. The first intensive investigation was carried out on the foundation subsoil by the Norwegian Geotechnical Institute (NGI, 1967) during the design and construction of highway starting in 1966. Data from trial embankments and properties of the soft clay subsoil were summarized by Holmberg (1984). Further geotechnical informations were obtained during the design and subsequent construction of the second carriageway from 1974 to 1975 by N.D. Lea and Thai Engineering Consultants (Cox, 1981; N.D. Lea et al, 1981). Recently, studies were also made by Parnploy (1985) and Pussayanavin & Leerakomsan (1986). A large portion of this paper were derived from the studies made by Ahmed (1986) under the guidance of the first author. A typical embankment cross-section and soil profile at km 30 are shown in Fig. 2.

Figure 1. Location map (after Cox, 1981).

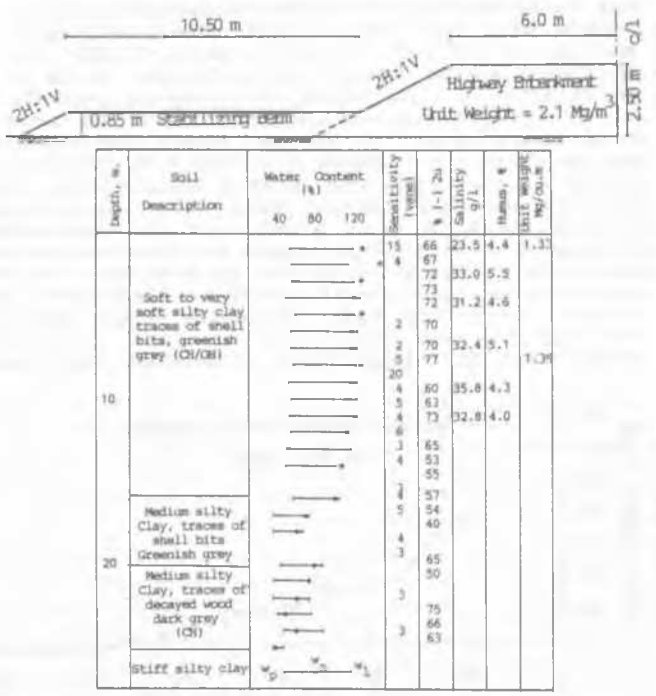


Figure 2. Typical embankment cross-section and soil profile at km 30 (after N.D. Lea et al, 1981)

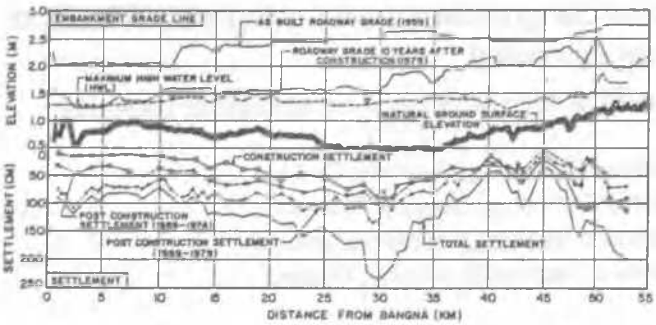


Figure 3. Ground elevations and settlement profile along the highway (after Cox, 1981).

2 SETTLEMENT CHARACTERISTICS ALONG THE BANGNA-BANGPAKONG HIGHWAY

In 1979, actual settlements were measured and a summary was presented by Cox (1981) as shown in Fig. 3. Settlement-time relationships for selected sections along the alignment were analyzed and studied. It was observed from field measurements that settlement rate have slowed down considerably in 1974. However, the settlement rates near Bangna at km 2+899 increased to 10 cm/yr instead of decreasing. This is blamed on the effect of Bangkok subsidence (AIT, 1982). Figure 4 shows the subsidence rates along the road alignment from 1979 to 1983. The ground subsidence is caused by excessive pumping of groundwater. Bangna area was one of the worst subsiding area in Bangkok with a subsidence rate of 10 to 15 cm/yr. The settlement rate between 1974 to 1979 increased as one proceeds from Bangpakong to Bangna inspite of the fact that the soil condition was stronger towards Bangna.

3 GEOTECHNICAL PROPERTIES OF SUBSOIL ALONG BANGNA-BANGPAKONG HIGHWAY

The whole stretch of the Bangna-Bangpakong Highway passes over the flat, deltaic plain of Thailand where the subsoil is soft marine clay whose thickness varies from 15 m at Bangna, to 25 m at km 28 (Fig. 1). The age of this marine deposit is about 2,000 years and is considered as recent deposit. The natural ground elevation in 1967 varies from 0.8 m above sea level at Bangna, to 0.6 m at km 30 to 1.2 m near Bangpakong. The natural moisture content generally exceeds the liquid limit in the first 5 m and decreases thereafter. The plastic limits are fairly constant with depth, being 30 to 35% except near softer areas in km 30 where it rises to 40 to 50%. The liquid limits are high, averaging about 100% in the top 5 m and increasing to 130% in softer areas. The undrained shear strengths from field

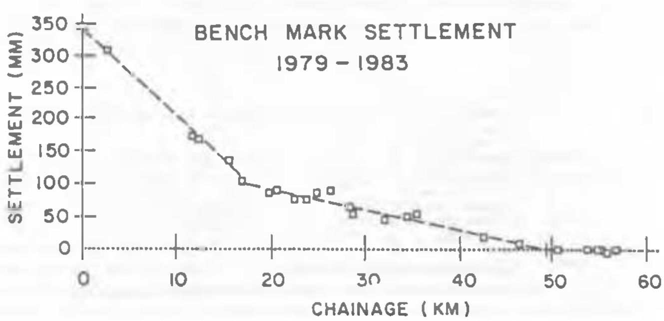


Figure 4. Settlement due to subsidence along the highway (after Pussayanavin & Leerakomsan, 1986).

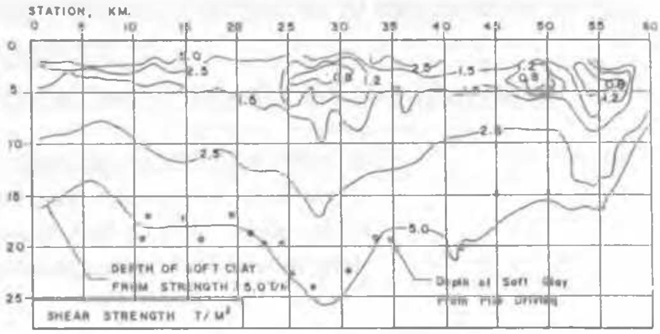


Figure 5. Undrained shear strength contours along the highway (after N.D. Lea et al, 1981).

vane are higher at the top 1 m, decreasing to about constant values near 5 m depth and then increasing with depth (Fig. 5). It can be observed that there are two soft stretches near km 30 and near km 50. Two main groups of subsoil are evident. The first group comprises the soft soils from km 20 to 35 and km 50 to 55 and the second group comprises the stronger soils from km 0 to 10 and 40 to 50. Transitory group of soils are found between km 10 and 20 and between km 35 to 40 which have compressibilities intermediate between the two main groups.

4 METHODS OF SETTLEMENT PREDICTION

4.1 Immediate Elastic Settlement

D'Appolonia et al (1971) developed a simplified method to account for local yielding beneath the embankment as the embankment approaches failure. The immediate elastic settlement can be calculated from the following formula:

$$s_i = q\ B\ I\ (1-\nu_u^2)\ /\ E_u \times SR \qquad (1)$$

where SR is the settlement ratio which is a function of the initial shear stress ratio (f), the applied stress ratio (q/q_{ult}), and the geometry of the problem. The value of E_u is normally expressed as a function of the undrained shear strength, S_u, i.e. $E_u = \alpha\ S_u$. The undrained Poisson's ratio, ν_u, is always taken as 0.5.

4.2 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 settlement since rapid dissipation of excess pore pressure resulting to an increase in effective stress, can be expected for an overconsolidated soil where the coefficient of consolidation, C_v, is very high. Thus, consolidation settlement during construction, s_{cc}, can be estimated by:

$$s_{cc} = \frac{C_r}{1+e_o} H \log \frac{\sigma_{vo}' + \Delta\sigma_v'}{\sigma_{vo}'} \ ; \ \sigma_{vo}' + \Delta\sigma_v' \leq \sigma_p' \qquad (2)$$

Cox (1981) adopted the combination of the above method and the method of D'Appolonia et al (1971) to estimate the construction settlement of the Bangna-Bangpakong Highway.

4.3 Consolidation Settlement

Skempton & Bjerrum (1957) developed a method in which the final primary consolidation settlement is calculated by correcting consolidation settlements computed from oedometer tests. The correction factor μ is a function of the pore pressure coefficient A and the geometry of the problem. For Bangkok soil, Lee (1983) obtained a chart showing the correlation of μ with OCR.

Davis & 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 (s_d) can be computed as:

$$s_d = \Sigma \frac{H}{E'} [\Delta\sigma_z - \nu'(\Delta\sigma_x + \Delta\sigma_y)] \quad (3)$$

Very few data are available for the drained modulus, E', but it is generally agreed that the value is some fraction of E_u. The drained Poisson's ratio, ν', lies between 0.30 to 0.45.

An observational settlement prediction was proposed by Asaoka (1978), 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 which is constant with time, the coefficient of consolidation, C_v, can be evaluated using the following relation:

$$C_v = -\frac{5}{12} H^2 \frac{\ln \beta_1}{\Delta t} \quad (4)$$

where β_1 is the slope of settlement records. 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:

$$s_c = \frac{H}{1+e_o} [C_c \log \frac{\sigma_{vo}' + \Delta\sigma_v'}{\sigma_p'} + C_\alpha \log \frac{t_1}{t_2}] \quad (5)$$

Reasonable agreement between actual settlement measured in 1979 and the estimated ultimate settlement from the above formula has been reached except in the first 20 km from Bangna where the effect of Bangkok subsidence is very high (Cox, 1981). Raymond & Whals (1976) proposed an equation to compute the amount of secondary compression.

5 INVERSE ANALYSIS

The method of back analysis used in this study may be termed as a combination of 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 using the curves of Olson (1977). Secondary settlement was then estimated and then taken out from the corresponding field measurement. Subsidence effect was also subtracted in the zone of km 0 to km 18. The remaining part (s_d) contains immediate (s_i) and consolidation (s_c) settlements only. The degree of consolidation just after construction (U_c) is then estimated.

2. Immediate settlement was obtained as a function of α from Eq. 1, where S_u was read off from Fig. 5 for each layer to estimate E_u. The subsoil was divided into three layers, namely: 0-5 m depth, 5-10 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 et al (1981). The consolidation settlement during construction (from 1967 to 1969) was then estimated as $s_c = (s_d - s_i)(U_c)$. The actual construction settlement (s_{cons}) was read off from the field-time settlement curves. The value of α is then evaluated from $s_{cons} = s_i + (s_d - s_i)(U_c)$. The drained settlement was estimated from Eq. 3 in terms of β ($\beta = E'/S_u$), where β is then evaluated by equating with s_d from step 1.

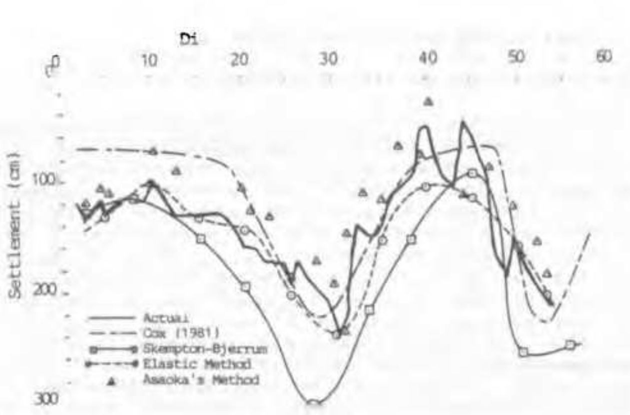
3. Representative values of α and β were chosen to carry out settlement analysis at selected sections (kms 5,10,15,...,45,50 and 53). Immediate and drained settlements were estimated, and U_c was evaluated from the knowledge of actual construction settlements. The corresponding value of T_c was obtained 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 $s_{total} = s_i + (s_d - s_i)(U) + s_{sec} + s_{sub}$, where s_{sub} is the subsidence settlement at the time of interest (where applicable). Secondary settlements were assumed to be small, and was 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.

6 ELASTIC PARAMETERS

The normalized value of E_u with respect to S_u for the whole stretch of the Bangna-Bangpakong highway were obtained. It was observed that α lies between 100 and 250 along the highway, which agrees well with the existing literature. 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. The mean and standard deviation of α obtained in this study were 168 and 67.3, respectively. A histogram was drawn for α and a representative value of 150 was recommended. The ratio of the drained modulus, E', and undrained shear strength, S_u, estimated for the 26 sections of the Bangna-Bangpakong highway were also computed. It can be observed that most of the points lie within the band between 15 and 40. The mean and standard deviation are 35 and 21.4, respectively. Further analysis of long term drained settlements, yielded β of 15 for the best estimation of field conditions.

7. COEFFICIENT OF CONSOLIDATION

The field values of coefficient of consolidation, C_v, estimated from the during-construction time-settlement behaviour of the embankment using Asaoka's method, were slightly higher as compared to the corresponding values estimated by Parnploy (1985) at km 2+899 of the main embankment. 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 and was 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. (2) Rapid consolidation close to the drainage boundaries, partly due to the effect of ground subsidence, leads to



Di
0
10
20
30
40
50
60
Settlement (cm)
100
200
300
Actual
Cox (1981)
Skempton-Bjerrum
Elastic Method
Asaoka's Method