Geotechnical design aspects for widening low embankments on soft ground

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ABSTRACT The proposed improvements to the existing Federal Highway Route 2 and the North Klang Straits Bypass involve widening and raising existing embankments over soft Klang clay. These pose a special problem which has to be overcome by geotechnical design : how to minimise differential settlements between the old embankment and the new fill, caused by the two parts settling at different rates. The results of analyses indicated that undesirably large differential settlements would occur across the finished embankment cross-section if the widening were to be carried out without special measures. A design was based upon partial precompression to eliminate only that part of the consolidation settlement that would make the settlement-time behaviour of the new fill match as closely as possible that of the old embankment.

1 INTRODUCTION

The widening of the existing North Klang Straits By-Pass (NKSB) and Federal Highway Route 2 (FHR2) embankments on soft marine clay involves the introduction of new fill over and on both sides of the existing embankment (Figure 1). Placement of additional fill will cause further settlement of existing embankments and, more significantly, large settlements under the new fill itself. The magnitudes and rates of settlement under the existing embankment and the new fill will be different, thereby causing differential settlements.

Several solutions are possible for eliminating or arresting the differential and total settlements of the embankments. However, not all of these solutions are economically viable or practical. For example, any "rigid" (eg. piled embankment) or semi-rigid (eg. stone column) solution would involve construction of the foundation system under the old embankment as well as the new embankment. This is not totally desirable as it would involve destruction and reconstruction of the old embankment, at least partially, if not completely. Construction of a totally new embankment has also been precluded due to the constraints of time and space in conjunction with the design criteria for the highway alignments and schedule of works.

This paper describes the analysis and design aspects that were adopted for widening existing low embankments on soft Klang marine clay for the Federal Highway Route 2 (FHR2) and a short leg of the North Klang Straits Bypass (NKSB) at the proposed NKSB Interchange. Low embankments are defined as those not exceeding 2.5 m in finished height above ground level.

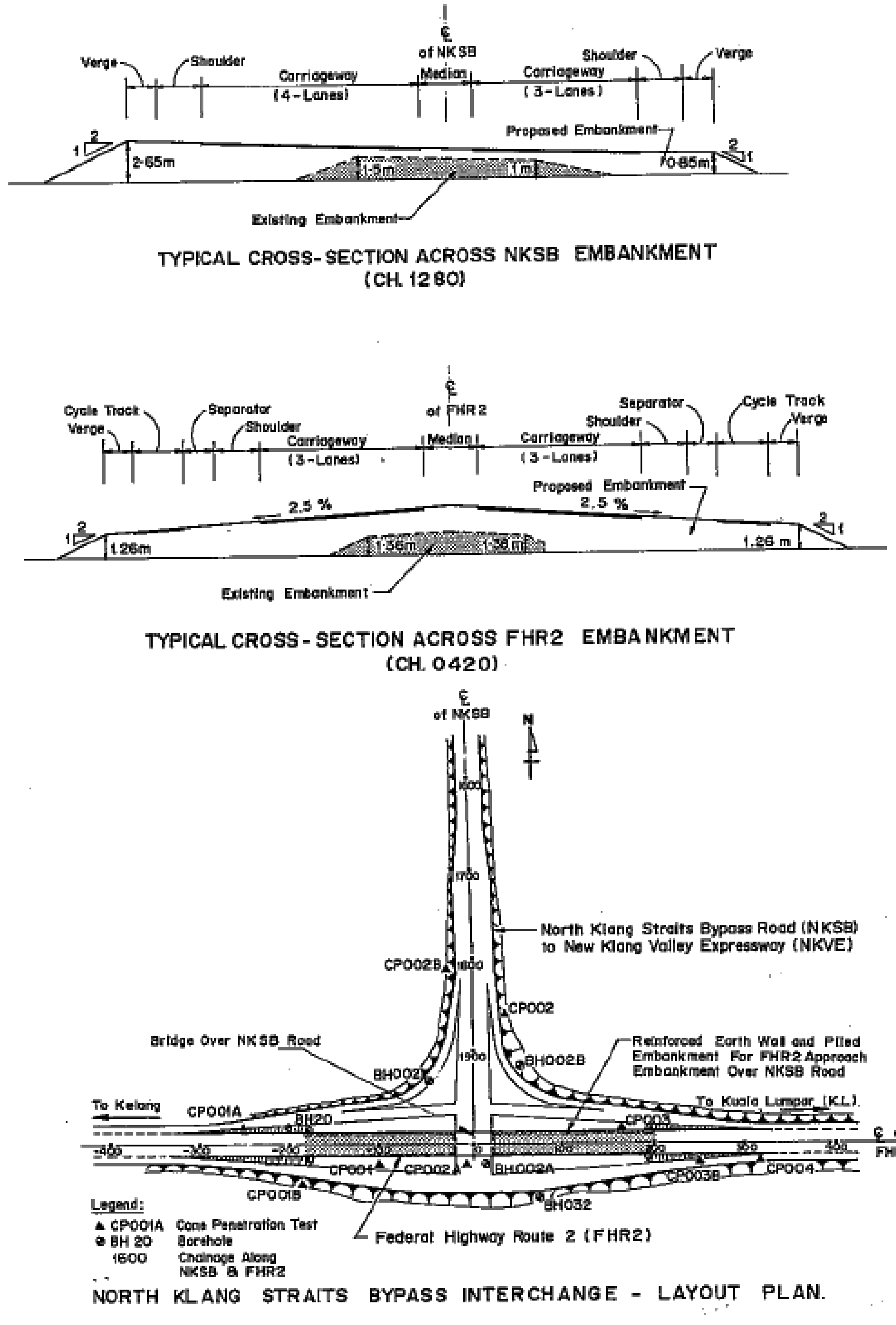

Figure 1 Site location and general arrangement of North Klang Straits Bypass Interchange

2 GEOTECHNICAL CHARACTERISATION

2.1 Geology

The proposed site is located east of Klang town on the eastern edge of the Selangor coastal plain. The Selangor coastal plain is situated between Sungai Bernam at the boundary between Selangor and Perak and the Devonian rocks 7km west of Kuala Sepang (Bosch 1988).

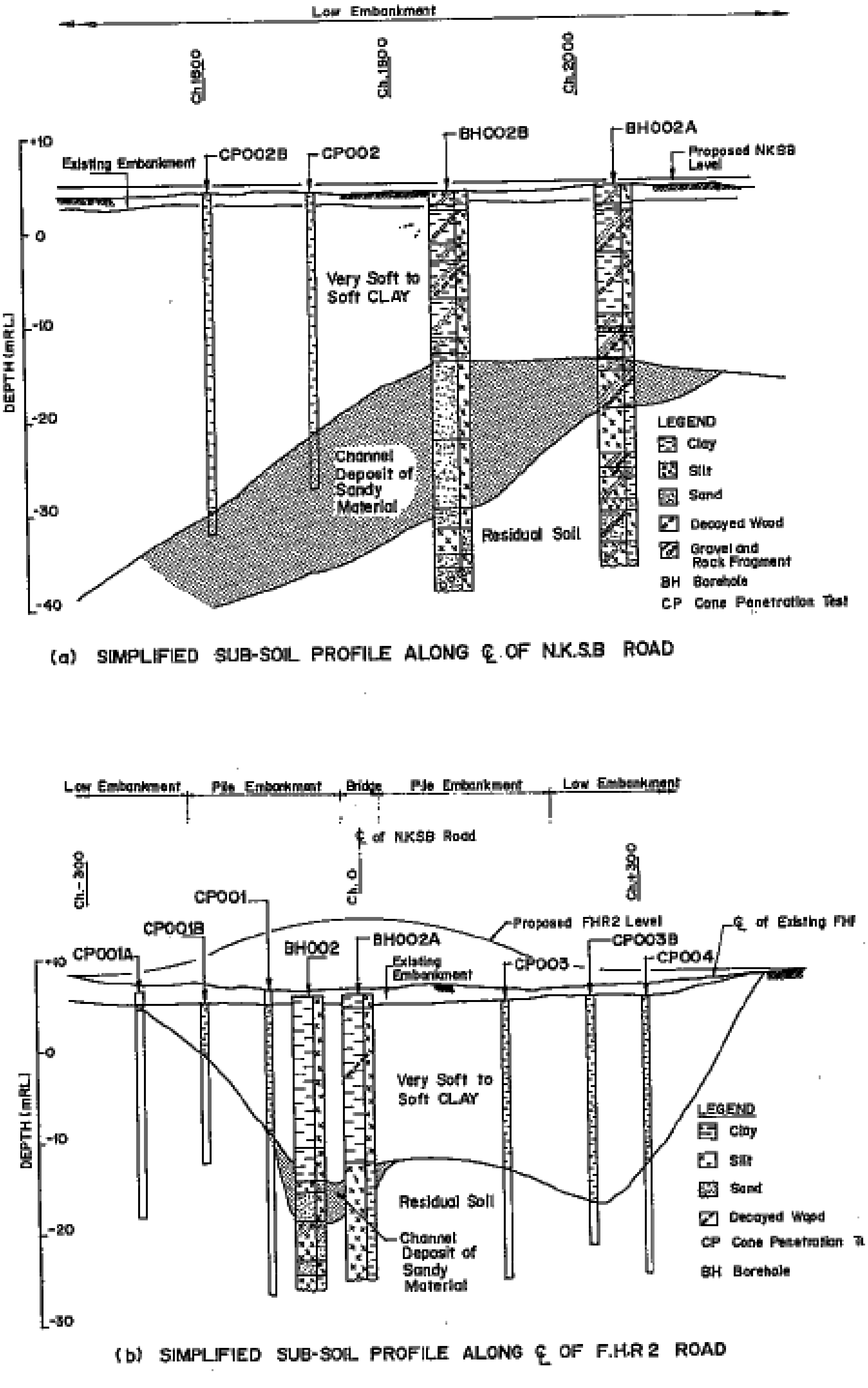

Figure 2 Sub-soil profile summary

Most of what is commonly known as "marine clay" by the geotechnical engineering community refers to the Holocene marine deposits of the Gula Formation and partly of the Port Weld Bed. The peats that exist along the west coast are of the Pengkalan Member of the Beruas Formation.

2.2 Site investigation

Preliminary site investigation was carried out on the FHR2 from the NKSB Interchange to Istana Bukit Kayangan and on the NKSB from April to July 1989. A second stage site investigation was performed from February to April 1990.

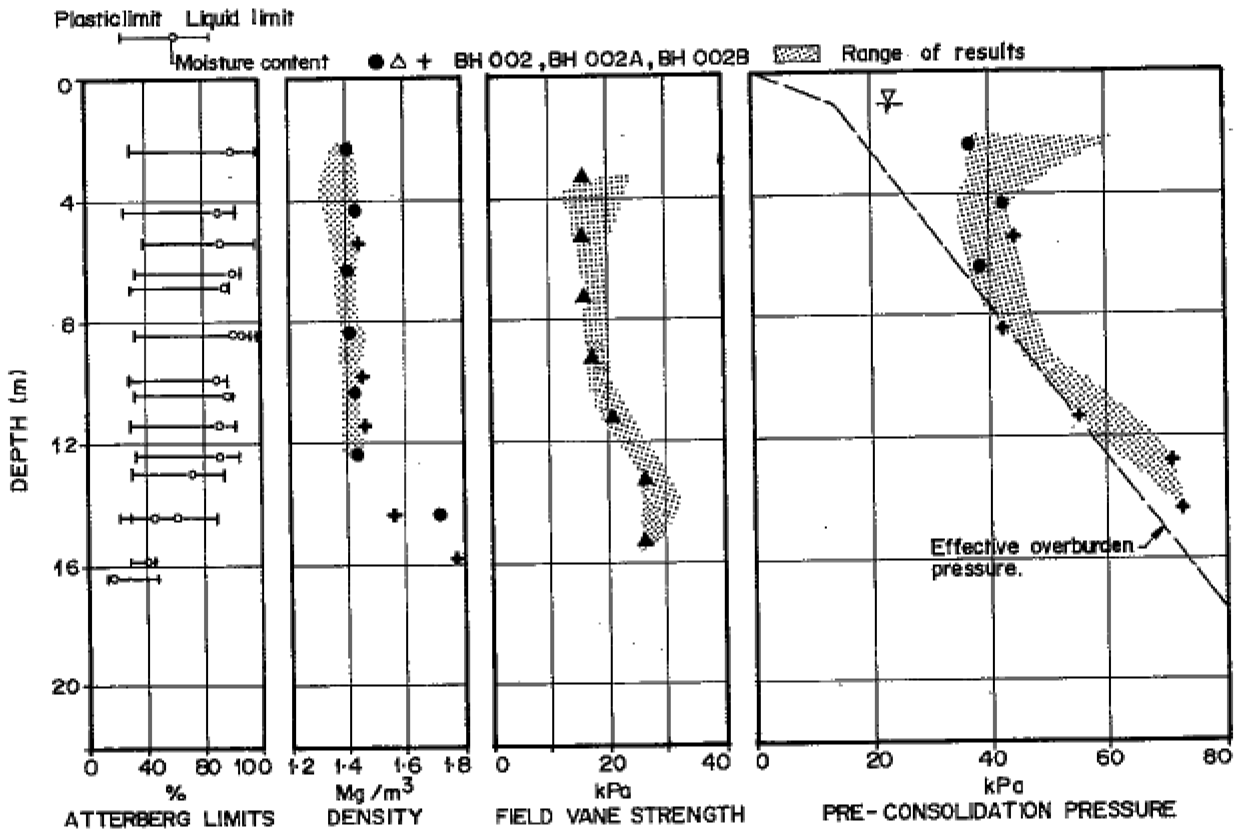

Figure 3 Typical engineering properties

The ground investigations revealed that the Klang coastal plain along the Federal Highway Route 2 is made up of very soft to soft silty clay with decayed wood, and is generally grey in colour. The thickness of these marine deposits is about 15 m to 20 m and gradually decreases eastwards from Klang. The underlying material is sandy silt and silty sand with traces of lateritic gravels at certain locations. Near the NKSB Interchange, a channel deposit of sandy materials was encountered.

Simplified sub-soil profiles along the Federal Highway Route 2 near the proposed NKSB interchange, and along a short leg of the North Klang Straits Bypass joining the Federal Highway Route 2 are shown in Figure 2.

2.3 Engineering properties

Typical engineering properties are presented in Figure 3. Bulk density appears to be fairly uniform with depth, and it appears reasonable to assume that it varies from 1.4 Mg/m^3 to 1.5 Mg/m^3. Initial void ratio, e_o, ranges from about 0.5 to about 3.5, with most values being between 2.0 and 3.0. Measured water contents were mostly close to the liquid limit, indicating high liquidity indices, and therefore high compressibility. The average values of the relevant Atterberg Limits were taken to be: LL $\simeq$ 115, PL $\simeq$ 45 and PI $\simeq$ 70. These values plot slightly above the A-line in Casagrande's plasticity chart, again indicating high plasticity and compressibility.

The parameters of utmost importance in view of the adopted solution were the preconsolidation pressure and coefficient of consolidation.

The values of the overconsolidation ratio (OCR) that can be deduced for the Klang soft clay are generally between 2 and 3 but are also as high as 4 in places within the top desiccated crust. Below this over-consolidated crust, the clay can be assumed to be lightly overconsolidated, with thickness of up to 12m at places.

In assessing the compression and consolidation characteristics of the soft clay, careful attention was given to the results of the oedometer tests below and above the preconsolidation pressures. Examination of the data indicated more consistency in the compression ratio, $C_c/(1+e_0)$, compared to the compression index on either side of this critical pressure. The compression ratio after the preconsolidation pressure was generally between 0.3 and 0.5, whereas an average of 0.07 was indicated for pressures below this threshold pressure. The values of the coefficient of consolidation vary within a range of 0.2 to 0.5 m^2/yr after the preconsolidation pressure. An extremely large variation of 2-70 m^2/yr was recorded for values below the preconsolidation pressure.

Most of the data on undrained shear strength were obtained from field vane tests (FVT). Although there are no FVT data within the top 3m, it is believed that the undrained shear strength within this overconsolidated top 3 m should be at least similar to, if not higher than, the undrained shear strength at about 3 m below ground level.

3 STABILITY

Stability analyses to determine safe embankment heights (Figure 4) were performed based upon the assumption of a circular failure surface through the foundation soil and neglecting the strength of the embankment fill materials. Based upon the latter assumption, the fill material was modelled as a surcharge in the analysis. The stability analysis did not take account of any increase in shear strength of the foundation soil due to consolidation.

A sensitivity analysis, by application of correction factors to the field vane test results, was also undertaken to finalise the selection of appropriate safe embankment heights.

4 SETTLEMENT ANALYSES OF LOW EMBANKMENTS

The maximum estimated immediate

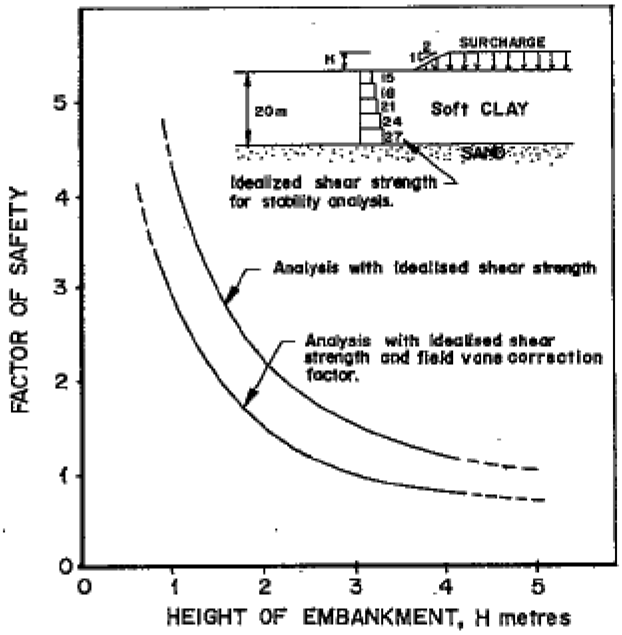

Figure 4 Results of stability analysis

settlement (D'Appolonia et al, 1971; Foot and Ladd, 1981) under the final embankment load is about 90mm under the NKSB embankments. This value of immediate settlement can be considered small, and is not expected to cause problems. Even under the surcharge loading the expected immediate settlements are approximately 130 mm, and can still be considered not excessive.

The consolidation analyses were performed using the computer program TCON. Secondary compression was not computed by TCON and was estimated in the conventional way.

In order to be meaningful, the analyses followed as closely as possible the loading sequence since construction of the old embankment, and then simulated the addition of the extra side fills at the appropriate point in time. Estimates of settlement versus time were undertaken at points of interest under the existing embankment and under the new added side fills. These positions were : under the centre of the existing embankment; under the edge of the existing embankment (i.e. the interface between old and new fill); and under the centres of the new side fills (Figure 5).

By estimating settlement at the above points

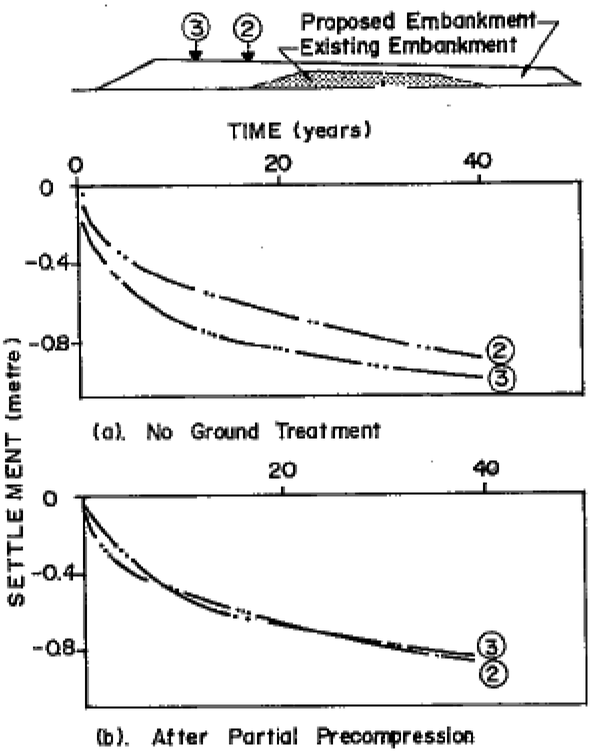

Figure 5 Embankment settlements (NKSB - CH 1280)

following construction of the existing (old) embankment, a history of expected differential settlements for a specified time period was simulated. For the purpose of the analyses, the NKSB was taken as having been constructed 5 years ago and the FHR2 20 years ago, which were the closest estimates of the construction schedule.

Of interest are the differential settlements that would occur after the present time if the embankment widening were to be performed without any special measures. In order to evaluate this, the settlement vs time curves were set to zero at t = 5 years for the NKSB and t = 20 years for the FHR2 and the analyses carried out accordingly.

Results of the analyses indicate that without ground treatment magnitudes of settlements vary appreciably along the stretches of the proposed embankments. For example, the differential settlements within 10 years across the old and new NKSB embankment range from 50mm at chainage 1620, which does not appear to be too excessive, to 200mm at chainage 1280 which would be unacceptable.

5 GROUND TREATMENT

The ground treatment solution adopted involves allowance for the new fill to settle at a rate matching as closely as possible with that of the old embankment. The method was to precompress the foundation soil under the new side fills to a condition such that the balance of the settlement versus time curve under the side fills matches the residual settlement versus time curve under the old embankment. This is termed partial precompression (Johnson, 1970).

Partial precompression can be considered practically viable with or without vertical drainage. In this particular case, vertical drains are not considered desirable. In the event of overlays being carried out in the future, the settlements under the old and new embankments would again occur at different rates, being faster under the new fill where vertical drains would exist. Thus, differential settlements would again arise across the embankment cross-section. Therefore, it was decided not to use vertical drains to accelerate precompression under the side fills. In any case, the analysis indicated that the time required for partial precompression to the desired degree in this particular case was attainable within the available construction time.

Partial precompression may be achieved with various combinations of surcharge load and time. Typically, surcharge thickness of 0.3m fill for a 4 months duration and a 1.45m surcharge height for a 6 month duration were adopted to produce the desired effect for the range of embankment heights to be constructed. Stability of the embankment with surcharge and practical considerations for construction, especially with respect to time and disturbance to traffic flow, were important governing factors for the selection of these preload (surcharge) combinations. Typical results of the proposed partial precompression scheme compared to the case of no treatment are presented (Figure 5).

6 MAINTENANCE ASPECTS

6.1 General considerations

The main limitation of the proposal for partial precompression is that there is no possibility of controlling the total settlements that will occur after construction is completed. This was realised and it was acknowledged that there must be a commitment to adopting higher maintenance associated with the solution.

Three types of problems are expected to arise after construction. These are :

(a) **Total Settlement :** the total settlements will in turn lead to loss of flood clearance and also loss of superelevation.

(b) **Differential Settlements :** although differential settlements have been minimised by design, they will not be completely eliminated. Also there may be locations where localised variations in conditions may lead to larger differential settlements.

(c) **Pavement Cracking :** this is a direct consequence of differential settlements. The areas where this is expected to be largest are at the interface between old and new embankments. A defensive measure was adopted by installing a continuous length of a nominal strength geopolymer under the pavement at this interface. This will minimize cracking, but will not prevent it completely if large differential settlements occur at localised spots.

6.2 Guidance for maintenance

It must be noted that the overlays themselves will add weight to the embankment which in turn induces further settlement. The settlement-time curve will also change with each overlay. It was therefore recommended that settlements be regularly monitored at selected points on the highway. Initial analysis indicated that annual overlays would be required for a 3 to 5 year period, primarily to even out differential settlements and to seal any pavement cracks which may develop.

7 CONCLUSION

The analysis and design of embankments over soft Klang marine clay for the NKSB interchange have been described. The special difficulties and constraints imposed by the requirements for widening a busy existing highway on soft ground are proposed to be overcomed by a partial precompression technique. In this approach, differential settlements of low embankments are minimised, but somewhat larger total settlements and a higher maintenance cost must be accepted.

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