FINITE ELEMENT ANALYSIS OF EMBANKMENTS ON SOFT CLAYS - CASE STUDIES

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ABSTRACT

In order to design structures on soft soils, it is necessary to predict the behavior of the soft soil under imposed structure load. The high excessive settlements of the soft soil can cause many problems for the structures built on the soil like cracking and breakup of pavements, railway, highway embankments, etc...

In this work, the finite element method is utilized as a tool for carrying out different analyses of embankments on soft ground with different conditions. The computer program CRISP (CRItical State Program) is developed to suit the problem requirements. CRISP uses the finite element technique and allows predictions to be made of soil deformations using the critical state theory.

Eight-node isoparametric quadrilateral element has been added to the program. The program was used to analyze fully coupled (Biot) consolidation of two-dimensional plane strain problems. The finite element predictions of displacements and excess pore water pressures were compared with field measurements.

It was concluded that the maximum vertical movement occurs below the centerline of the embankment. The settlement decreases slightly as the toe of the embankment is approached and decreases rapidly as the distance away from the toe increases. Upward movement of the surface far from the toe is observed. The maximum horizontal movement occurs near the top boundary. The rate of horizontal movement at the top of the foundation is greater than at the bottom. This behavior may be due to the flexibility and free movement condition of the vertical boundary in the top half

التحليل بالعناصر المحددة للتعليات الترابيية على تربة طينية رخوة – دراسات حالة

الخلاصة

لغرض تصميم المنشات التي تستند على ترب رخوة, من الضروري التنبوء بسلوك هذه الترب تحت تأثير احمال المنشأت. ان الهبوط المفرط العالي للترب الرخوة قد يؤدى الى حدوث مشاكل للمنشات المشيدة عليها مثل التشققات و تكسر طبقات التبليط و خطوط السكك و الطرق السريعة و السداد الترابية (التعليات) و غيرها.

في هذا البحث, استعملت طريقة العناصر المحددة كطريقة لاجراء عدد من التحليلات لسداد ترابية (تعليات) مشيدة على ترب رخوة و لحالات مختلفة. و قد طور برنامج الحاسبة المسمى (CRISP) ليناسب متطلبات المسالة. ان برنامج (CRISP) يستعمل تقنية العناصر المحددة و يسمح باجراء تخمين للتشوهات التي تحدث في التربة اعتمادا على نظرية الحالة الحرجة. و قد اضيف عنصر ثماني العقد رباعي الشكل إلى البرنامج. و استعمل البرنامج لتحليل مزدوج للانضمام حسب نظرية بيوت لمسائل ثنائية الابعاد ذات انفعال مستوي و قد قورنت نتائج طريقة العناصر المحددة للازاحات و ضغط ماء المسام الاضافي مع القياسات الحقلية .

لقد وجد أن الإزاحة الشاقولية العظمى تحدث تحت الخط المركزي للسداد (التعلية) و تتناقص هذه الازاحة كلما تم الاقتراب من حافة السدة ثم تتناقص بشكل سريع عند الابتعاد عن حافة السدة و قد لوحظ حدوث ازاحات شاقولية للأعلى لسطح التربة الواقع بعيداً عن جسم السدة (التعلية).

كما تحدث الازاحة الافقية العظمى بالقرب من الحدود العليا لطبقة تربة الاساس. ان معدل الازاحة الافقية في النصف العلوي من الاساس يكون اعلى من قيمته في النصف السفلي. ان هذا التصرف يمكن ان يعزى الى مرونة الحركة و الشرط الحدودي الحر للسطح العلوي من الاساس.

KEY WORDS
Finite Elements, Embankment, Soft soil, Cam Clay, Consolidation analysis.

INTRODUCTION

The increase in the world population and the consequent demand on new areas for construction forced the civil engineer to construct on large lands that have been avoided in the past because of their inadequacy as foundation ground. This means that land which was formerly considered unsuitable has now been utilized by building over. However, before this can be done, the problems which made the land unsuitable in the first instance have to be overcome by the engineer either by special design or ground improvement.

In order to design structures on soft soils, it is necessary to predict the behavior of the soft ground under impose of structure load. The high excessive settlements of the soft soil can cause many problems for the structures built on the soft soils like cracking and breakup of pavements, railway, highway embankments, etc...

The engineering properties of the soft clay may be improved to make the soil suitable for construction; several techniques have been suggested to improve the engineering properties of the soft soil.

PROBLEMS IN PREDICTING SETTLEMENT OF EMBANKMENTS ON SOFT CLAY

The major uncertainties in predicting settlement of embankments on soft clays are list:

1. Uncertain stratigraphy and drainage boundaries.

2. Uncertain time history of loading.
3. Mass movements due to shearing stresses.
4. Economical problems that restrict exploration, testing, and other analyses.

Other problems that clearly influence the accuracy of predicting include (Olson and Fellow, 1998):

1. Effects of secondary compression. Field data documenting long-term behavior are required. Many field observations terminate when a structure (pavement, bridge, tank, etc...) is constructed, because the instrumentation is destroyed. The client is not interested in paying for continued readings, and the engineer does not wish to risk for being used based on the documented movements.
2. Sampling disturbance compared with disturbance introduced in the field such as during wick installation.

Use over simplified and analytical procedure, or performing the design based on "Engineering Judgment" with minimal exploration and testing, is an effort to underbid the complication.

THE COMPUTER PROGRAM CRISP

CRISP (CRItical State Program) was developed at Cambridge University, Engineering Department, Soil Mechanics Group, in 1975. Later after making necessary modifications on (CRISP), it was republished in 1987.

CRISP uses the finite element technique and allows predictions to be made on ground deformation using critical state theories.

CRISP is used in this work for the analysis of embankment problems after making necessary developments. These developments include the following (Al-Shammary, 2006):

1. Eight-node isoparametric quadrilateral element is added with all necessary modifications in corresponding matrices.
2. Some subroutines are rearranged and organized.

Summary of Facilities:

Solution techniques:

There is a number of techniques for analysis of non–linear problems using the finite elements. CRISP uses the incremental or tangent stiffness approach. The user divides the total load acting into a number of small increments and the program applies each of these incremental loads in turn. During each increment, the stiffness properties appropriate for the current stress level are used in the calculations.

Excavation, Construction and Increment Blocks:

A finite element program intended for geotechnical analysis should be capable of analyzing problems where soil is excavated or soil structures (e.g. embankments) are constructed. This is not a standard feature found in finite element programs in other branches of engineering. CRISP allows elements to be removed to simulate excavation and elements to be added to simulate construction. The applied loadings for these cases are automatically calculated by the program.

When performing a non-linear analysis involving excavation or construction, the requirement for relatively small applied loads in each increment still applies. The way of achieving this is the removal or addition of a large number of layers of "thin" elements due to the large number of elements, and possible numerical conditioning problems associated with elements that have large aspect ratios. CRISP circumvents this problem by allowing the effect of element removal or addition to be spread over several increments in an "increment block" which is a series of ordinary increments grouped together in the input data for the program.

CASE STUDY PROBLEMS

The construction of embankments on soft clays will be simulated by using the eight-node isoparametric elements.

SIGMA Test Embankment Problem

This is one of the basic problems of the computer program GeoSlope. In this problem, the subsurface clay stratum is 9 m deep and the water table is 1 m below the ground surface.

The 1 m-layer above the water table is highly weathered desiccated and fissured making the behavior similar to a fine granular soil.

The embankment to be analyzed is shown in Fig. 1 and is constructed of relatively sandy soil. The height of the embankment is 5 m with 3:1 side slopes and 10 m crest width. Due to symmetry about the center, only half of the cross section is modeled in the analysis. Part of the underlying soil (clay) is considered in the analysis.

The bottom of the problem is fixed, while both vertical ends are allowed to move vertically but not horizontally. The finite element mesh of the embankment and its foundation is shown in Fig.1

Clay Properties

The clay is modeled as a modified Cam –clay (MCC). It is highly plastic with liquid limit (LL) of 61%. The compression index, Cc, can be estimated from the well-known equation (Skempton, 1944):

$$C_c=0.009*(LL-10 \text{ percent}) \tag{1}$$

The slope of the normal consolidation line, λ, can be computed from $\lambda = C_C/2.303$ (Atkinson and Bransby, 1978). A summary of the soil properties is provided in Table 1.

Sand Properties

The embankment and the upper one meter of subsoil is modeled as highly permeable. Linear elastic model can be used for this material, which can be considered adequate since the major settlement issue arises in the underlying compressible clay. The properties adopted for sand are also given in Table 1.

Table 1 The properties of foundation and embankment of problem (Manual of the program Geo-Slope).

Type of soil	Model	Parameter	Value
Clay	MCC	Liquid limit (LL)	61%
		Compression index (C_C)	0.46
		Slope of normal consolidation line (λ)	0.2
		Slope of swelling line (κ)	0.04
		Coefficient of volume compressibility (m_v)	$3*10^{-4}$ (1/kPa)
		Hydraulic conductivity (k)	$5*10^{-3}$(m/day)
		Effective friction angle (ϕ)	26°
		Slope of critical state line (M)	1
		Coefficient of earth pressure at rest (K_0)	0.56
		Poisson's ratio (ν)	0.36
		Over-consolidation ratio (OCR)	1.2
		Unit weight (γ)	20 kN/m^3
Sand	Linear elastic	Modulus of elasticity (E)	2000kPa
		Poisson's ratio (ν)	0.36
		Hydraulic conductivity (k)	1 m/day
		Coefficient of volume compressibility (m_v)	$3*10^{-4}$ (1/kPa)
		Unit weight (γ)	20 kN/m^3

<u>In Situ-Stresses</u>

An essential feature when using the modified Cam clay (MCC) soil model is to establish the initial yield surface. Any changes due to the embankment loading are relative to the yield surfaces that exist prior to loading. The initial yield surface is related to the initial in-situ stress and over–consolidation ratio (OCR). The (OCR) is specified as a soil property but the in-situ stress must be computed in a separate step.

The total and effective horizontal stress profiles in the sub-soil before starting the embankment fill placement are shown in Fig.2.

<u>Loading sequence</u>

The embankment fill is going to be placed in five stages as follows:

Fill lift	Elapsed time (days)	Height (m)
1st	1	1
2nd	7	2
3rd	13	3
4th	19	4
5th	25	5

Results and Discussion

Comparison of Results

Fig. 3 shows a comparison between the settlements that occur at the centerline at the water table elevation predicted by the CRISP program which is used in this study and those by SIGMA program. It can be seen that the settlement predicted by the two programs is similar during construction of the test embankment. The excess pore water pressure changes with time at a specified location one meter to the right of center line and two meters below the water table (point B in Fig. 1) are compared in Fig. 4 for both programs. The SIGMA program shows lesser excess pore water pressure during construction, while, after construction, the two results tend to be similar.

The pore water pressure starts to dissipate immediately after construction of each fill lift, then its value increases when the next fill is constructed and dissipation continues.

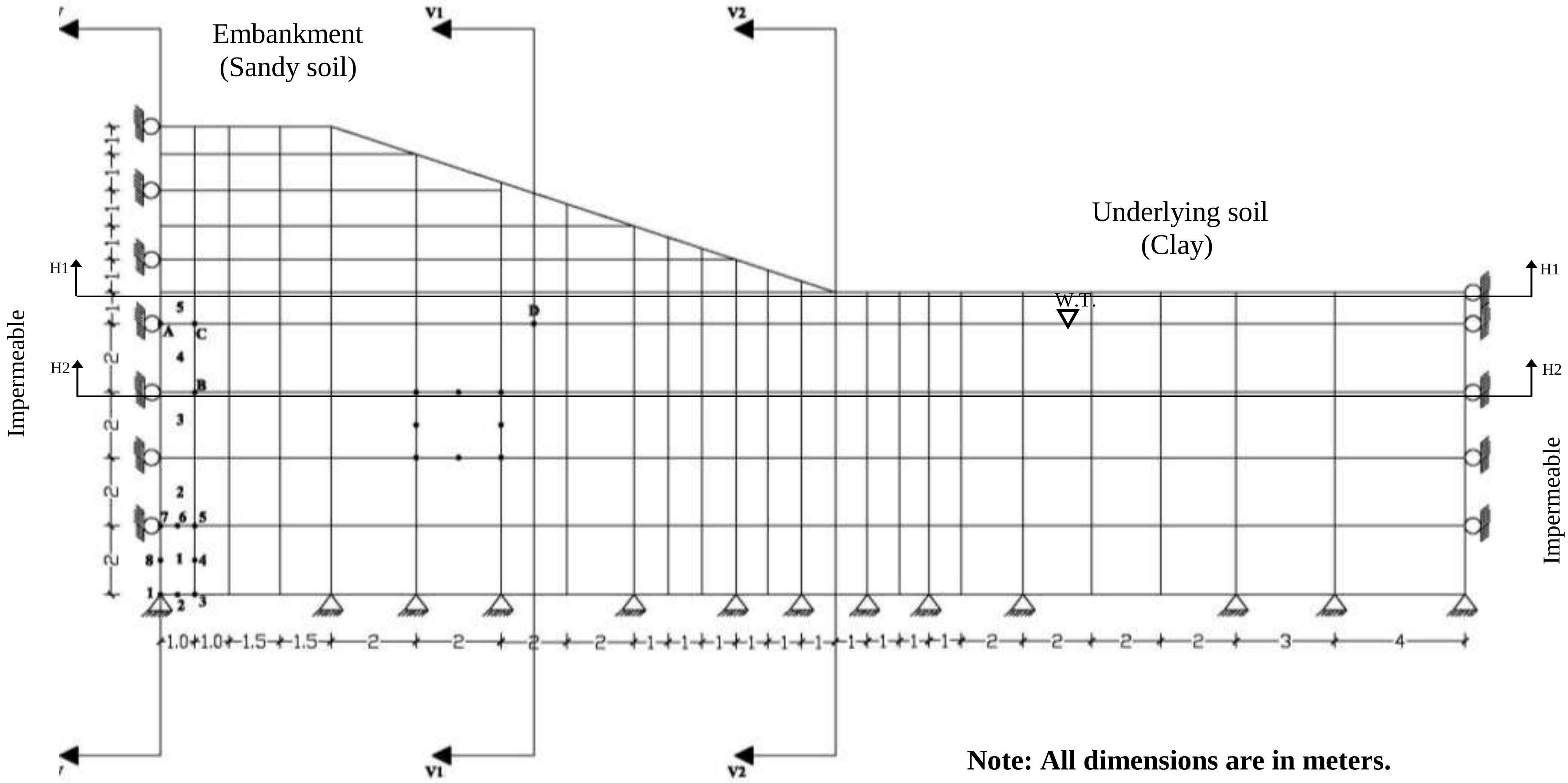

Fig.1 The finite element mesh of SIGMA embankment.

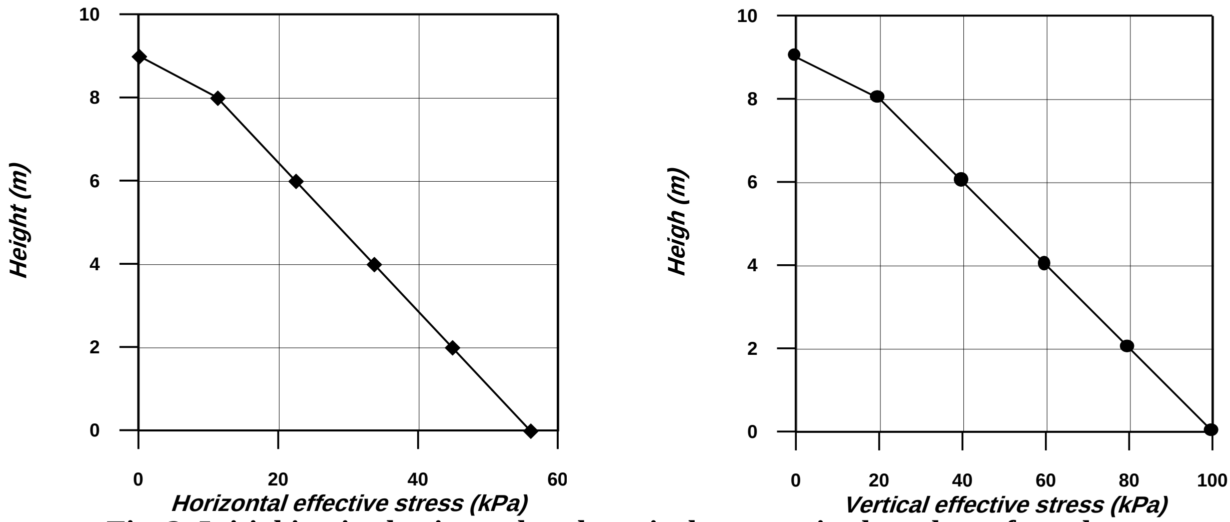

Fig. 2 Initial in-situ horizontal and vertical stresses in the sub-surface clay.

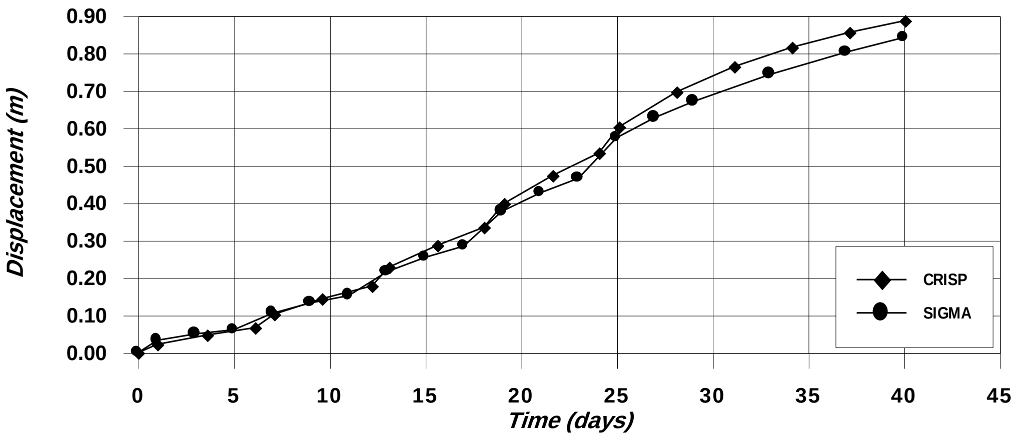

Fig. 3 Vertical displacement at node (A) of SIGMA test embankment.

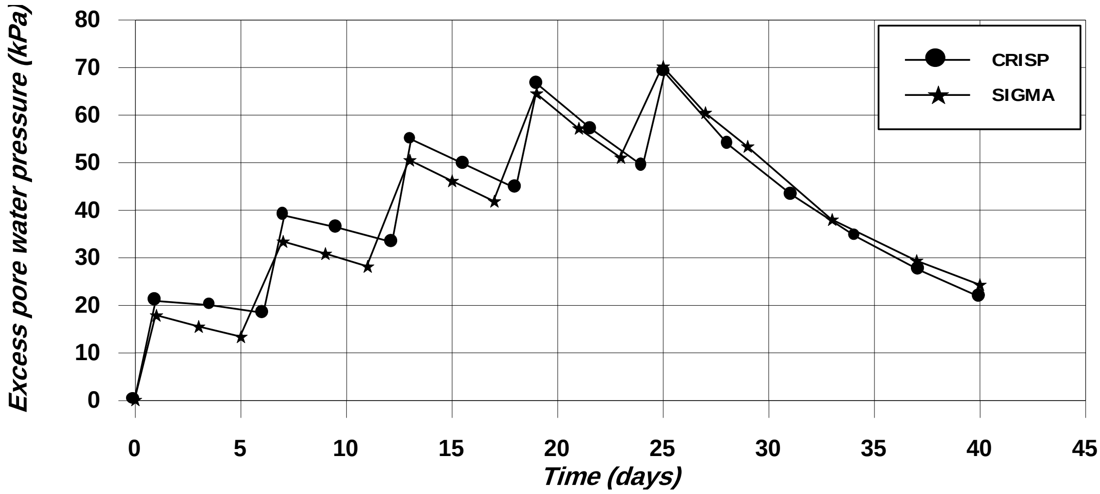

Fig. 4 Excess pore water pressure at node (B) of SIGMA test embankment.

Vertical Displacement

In Figures (5 and 6) the finite element analysis results of the vertical displacements along horizontal sections (H1-H1), (H2-H2) (as shown in Fig. 1) during construction are presented. The maximum settlement does not occur along the centerline but is located more towards the outer edge of the fill in the early stages of loading. This behavior is due to the zone of lower effective horizontal stresses. In addition, the toe of the fill has been lifted up slightly because the zone near the toe is a high shear stress zone. Upward movement of the surface far from the toe is observed due to the large load applied at the centerline of the foundation. This upheaval decreases slightly towards the toe of the embankment and decreases significantly as the distance away from the toe becomes large.

The vertical settlement along the vertical sections (V1-V1) and (V2-V2) and the centerline are shown in Figures (7, 8 and 9). From all these figures, the maximum vertical displacement was found at elevation 9 m because this level will be exposed to load directly. After construction, the maximum settlement will occur at the centerline as shown in Figs. (10 and 11). This behavior is due to the fact that the zone near the center of embankment suffers from concentration of stresses. In Fig. 9, the maximum settlement is found at elevation 4 m because this region is affected by all strains developed above this level resulted from the loads applied above this level. On completion of the embankment, the staged construction is halted and the load remained constant. In this stage, the consolidation of the soil will occur without any increase in the applied load.

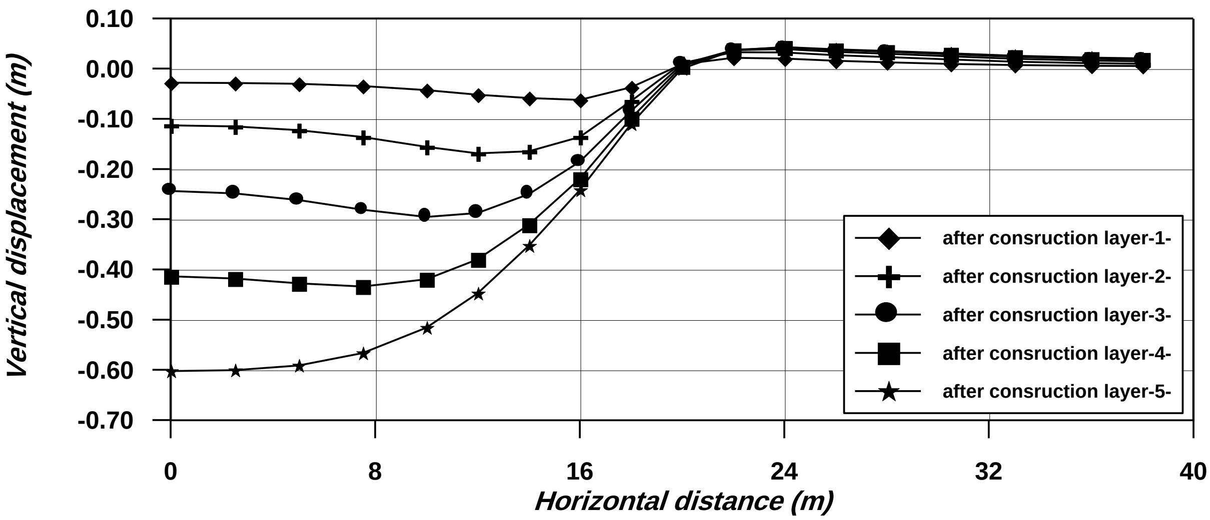

Fig. 5 Vertical displacement along section (H1-H1) during construction of SIGMA test embankment.

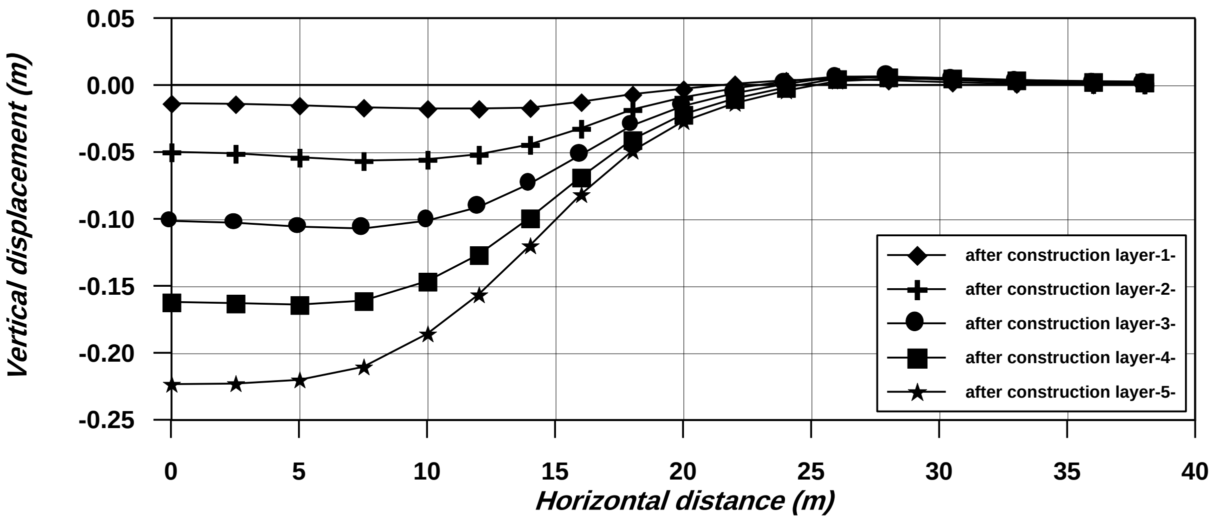

Fig.6 Vertical displacement along section (H2-H2) during construction of SIGMA test embankment.

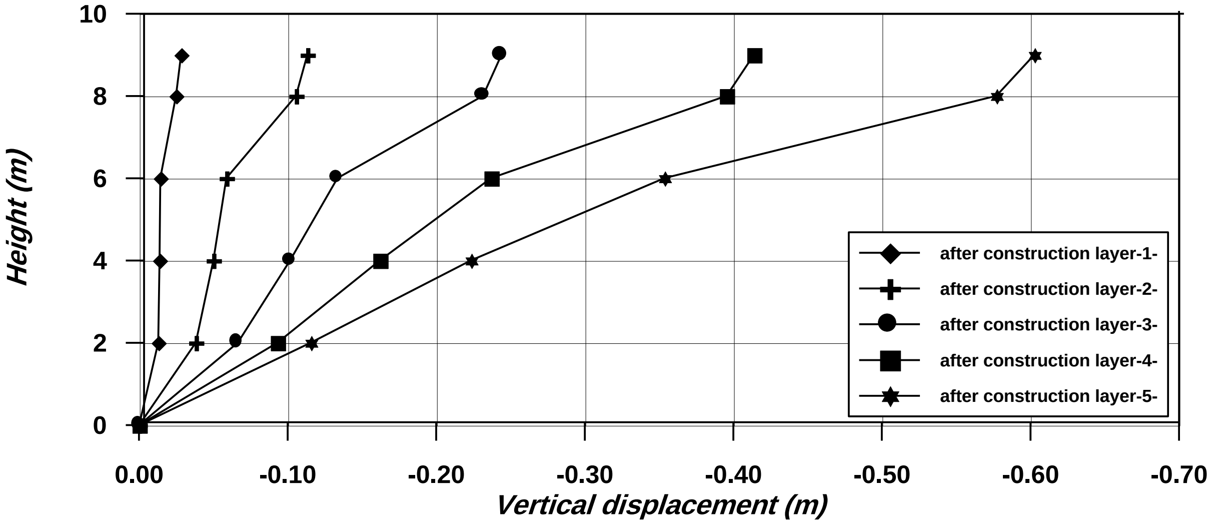

Fig.7 Vertical displacement along the centerline during construction of SIGMA test embankment.

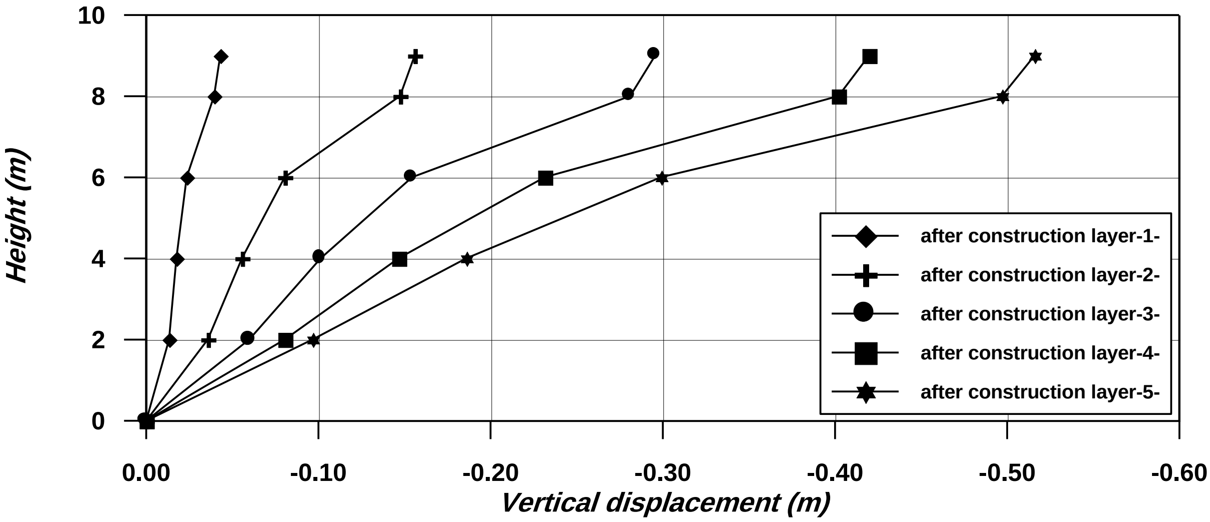

Fig. 8 Vertical displacement along section (V1-V1) during construction of SIGMA test embankment.

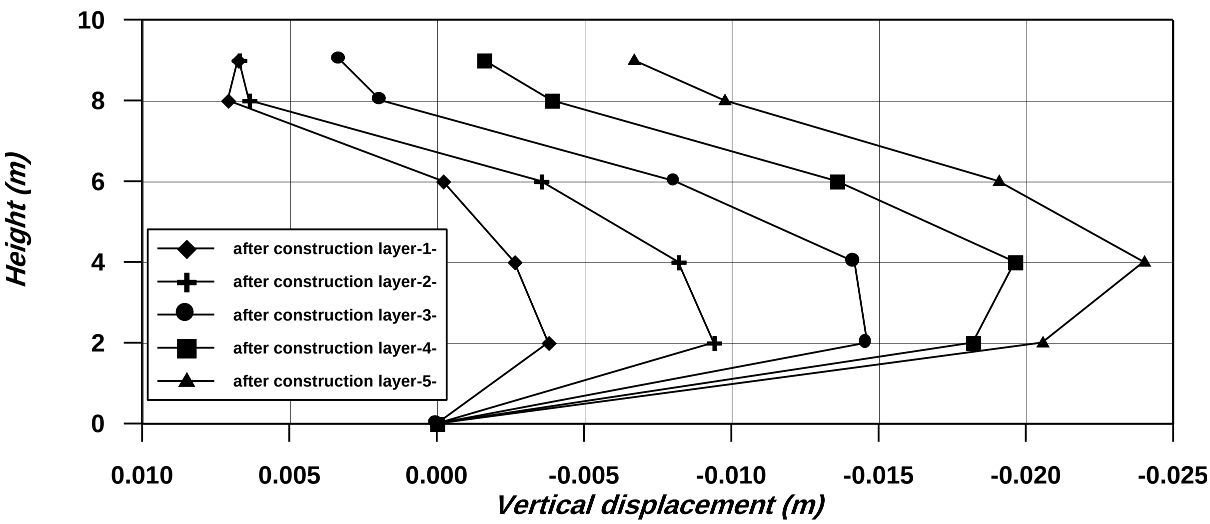

Fig. 9 Vertical displacement along section (V2-V2) during construction of SIGMA test

embankment.

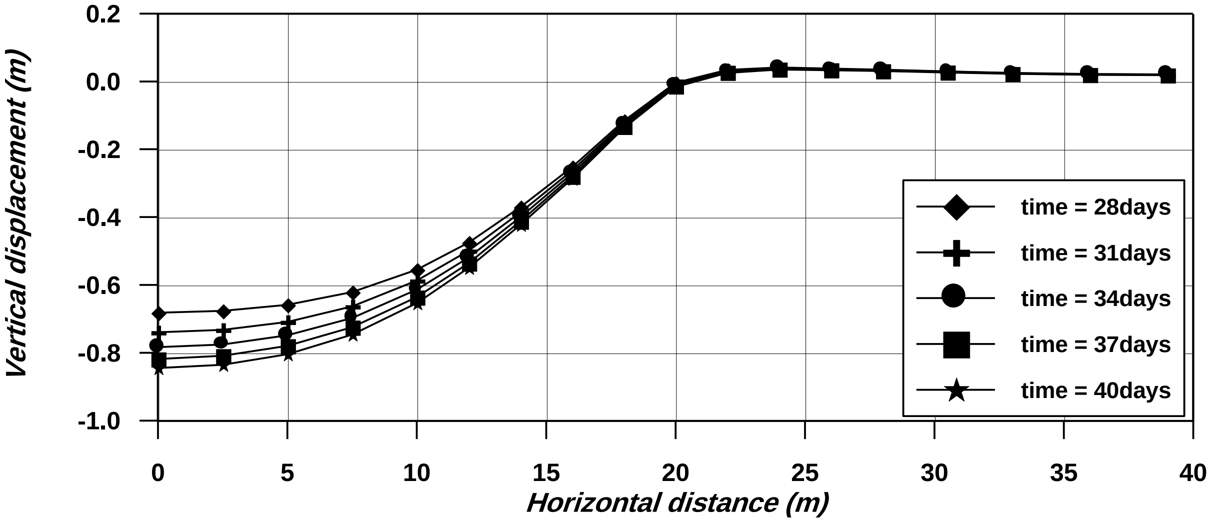

Fig. 10 Vertical displacement along section (H1-H1) after construction of SIGMA test embankment.

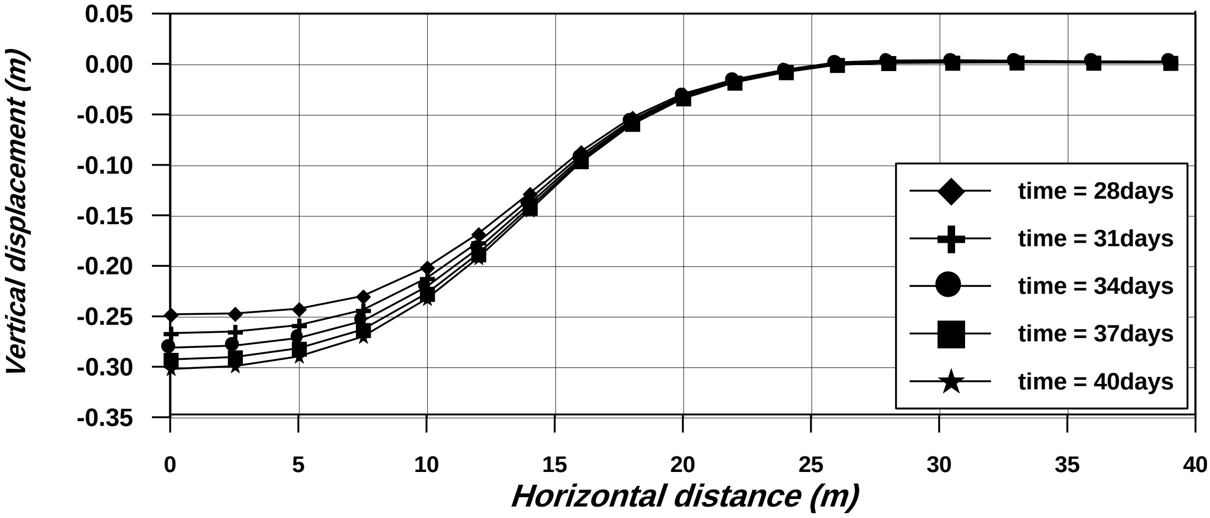

Fig. 11 Vertical displacement along section (H2-H2) after construction of SIGMA test embankment.

Horizontal Displacement

The horizontal movements along two vertical sections, (V1-V1) and (V2-V2) are presented in Figures (12 and 13) during construction. In general, it is seen in these figures that the maximum horizontal displacement occurs near the top boundary. The rate of horizontal movement at the top of the foundation is greater than that at the bottom. This behavior may be due to the flexibility of the top boundary assumed for horizontal movement. It can also be seen that section (V1-V1) near the toe shows the maximum horizontal displacement in comparison with the other section (V2-V2). This behavior is due to the large difference between the vertical load concentration on the left and right hand sides of section (V1-V1).

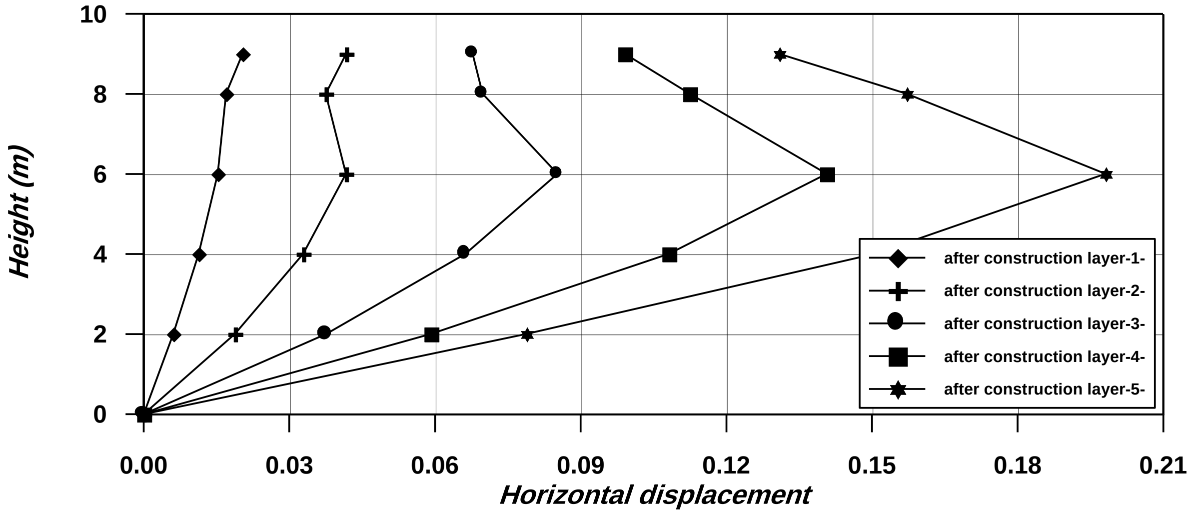

Fig. 12 Horizontal displacement along section (V1-V1) during construction of SIGMA test embankment.

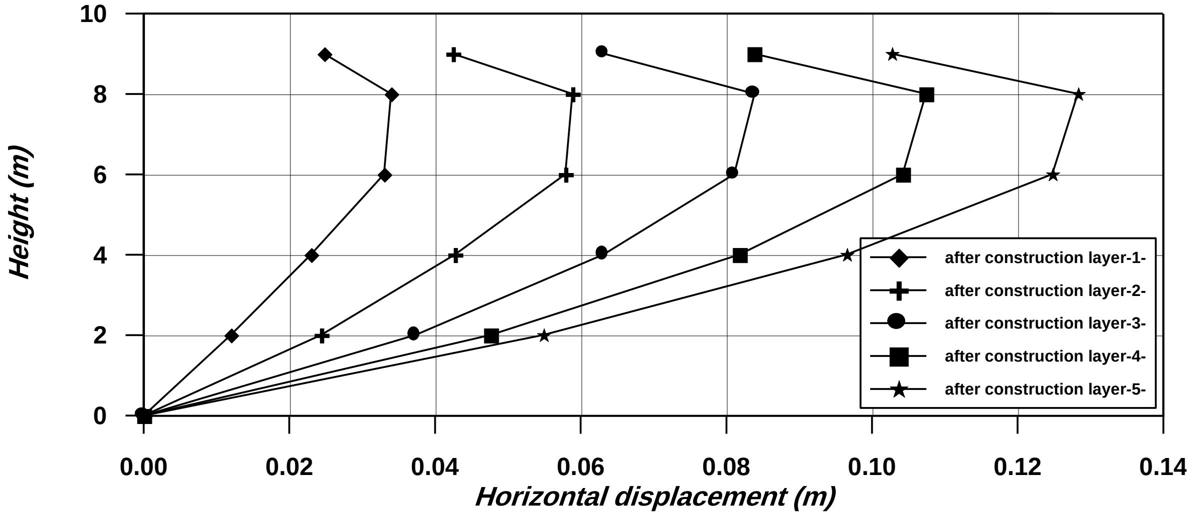

Fig. 13 Horizontal displacement along section (V2-V2) during construction of SIGMA test embankment.

Excess Pore Water Pressure

The excess pore water pressure dissipation along the centerline, sections (V1-V1) and (V2-V2) are shown in Figures (14 to 19). The curves show the isochrones of excess pore water pressure during construction and after construction at times 28, 31, 34, 38 and 40 days. As it is shown, the isochrones of excess pore water pressure give symmetric curves with maximum values near the middle of the foundation soil. This behavior is due to the two–way drainage assumption. The analysis predicts pore water pressure at centerline of the embankment greater than at the other two sections. This behavior is expected and it is due to the concentration of the load at the centerline of the embankment.

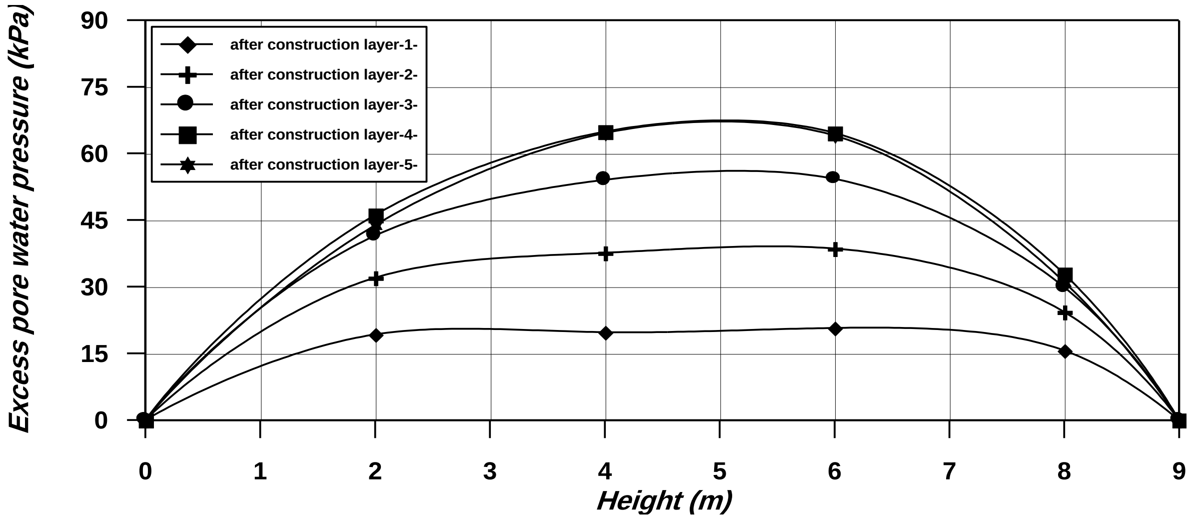

Fig. 14 Excess pore water pressure along the centerline during construction of SIGMA test embankment.

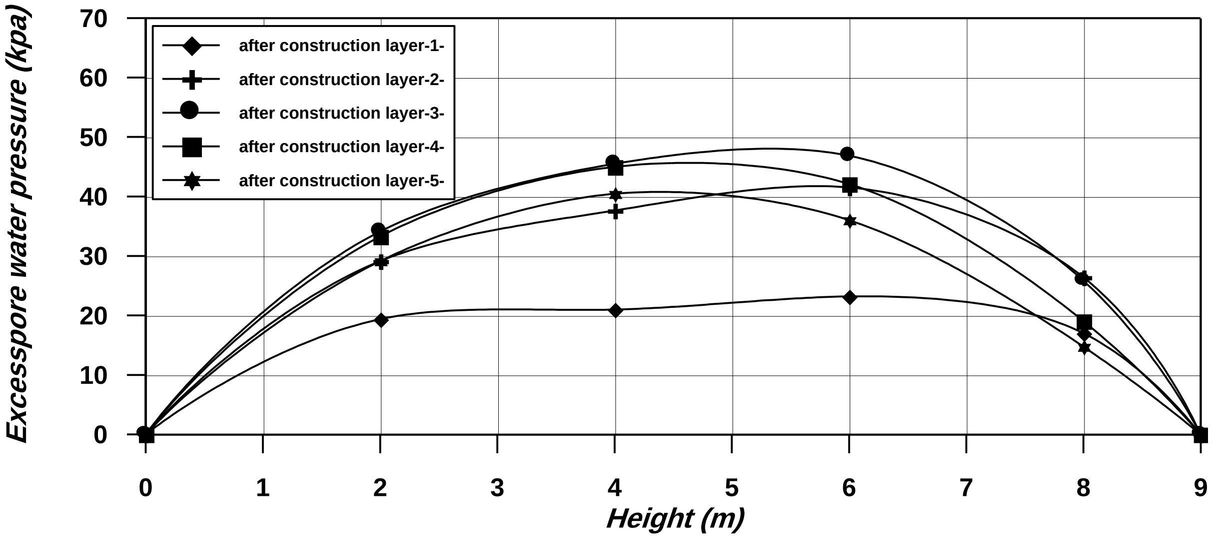

Fig. 15 Excess pore water pressure along section (V1-V1) during construction of SIGMA test embankment.

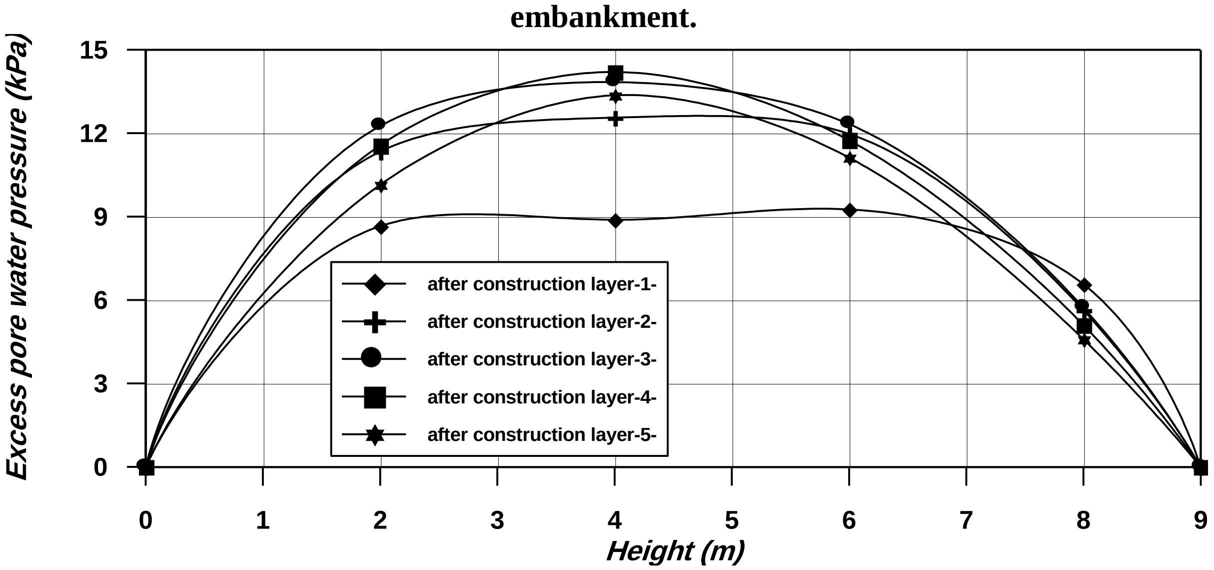

Fig. 16 Excess pore water pressure along section (V2-V2) during construction of SIGMA test embankment.

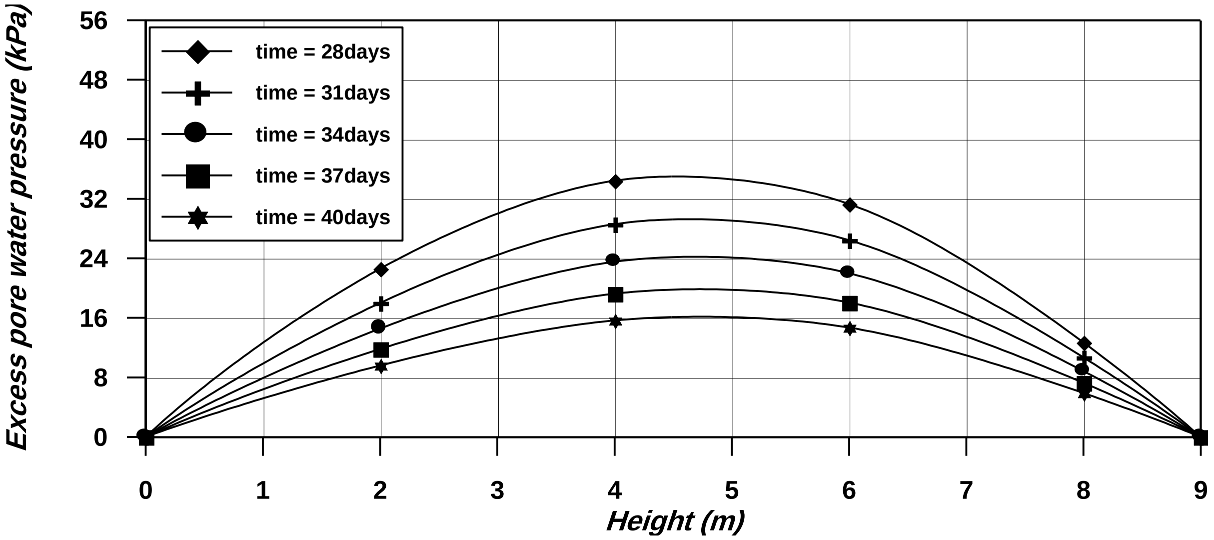

Fig. 17 Excess pore water pressure along section (V1-V1) after construction of SIGMA test embankment.

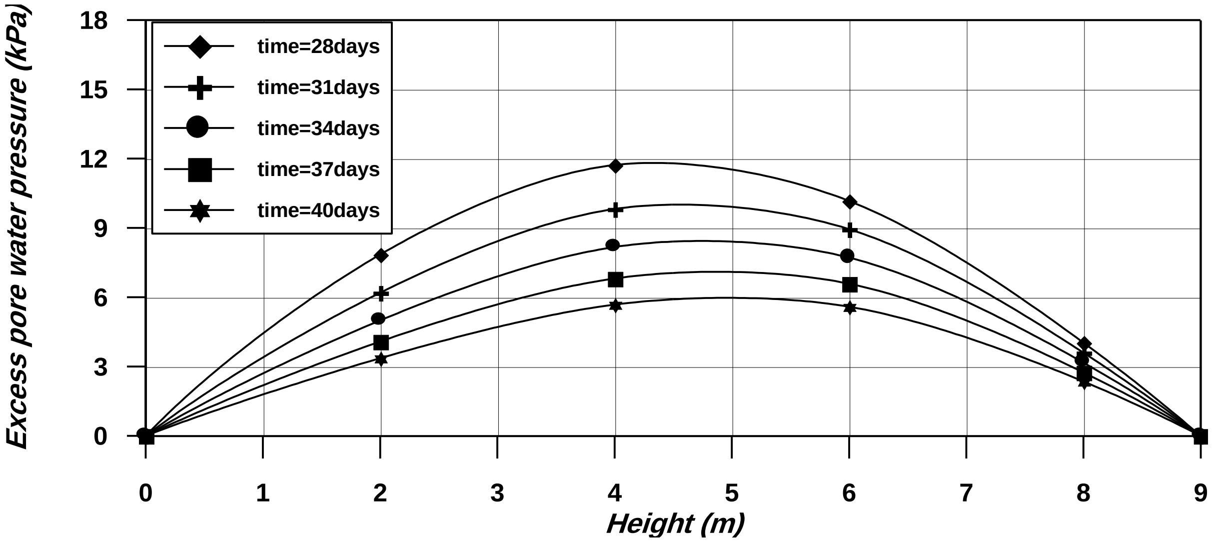

Fig. 18 Excess pore water pressure along section (V2-V2) after construction of SIGMA test embankment.

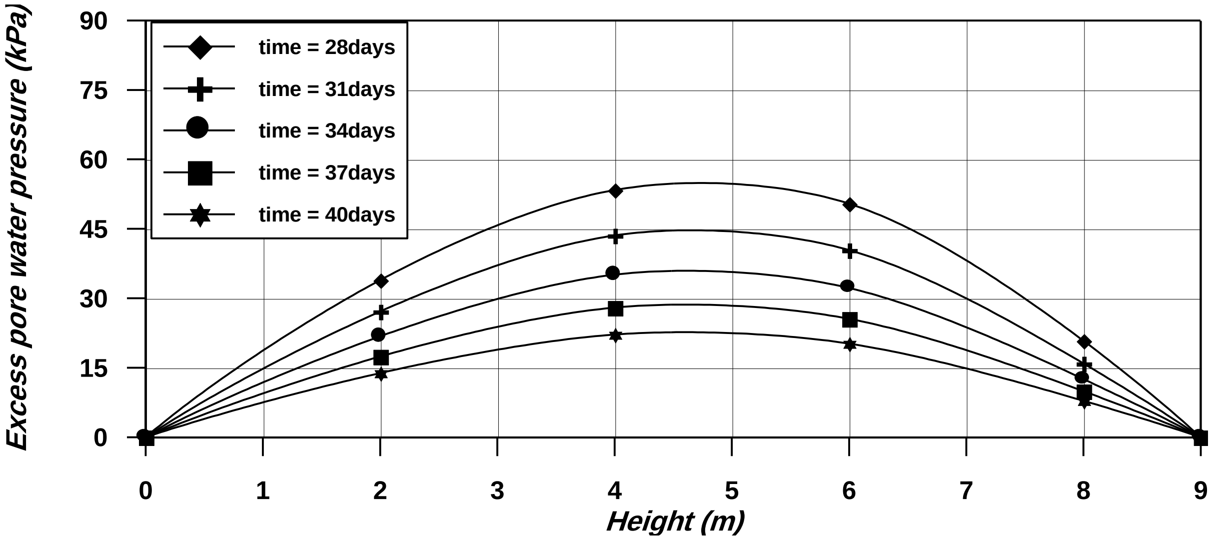

Fig. 19 Excess pore water pressure along centerline after construction of SIGMA test embankment.

Skä Edbey Embankment

Project Description: -

The test embankment was built on the site of Skä Edbey about 25 km west of Stockholm in the year 1961, to study the long-term behavior of Swedish clays and the suitability of the site for construction of an airport.

The embankment has a height of 1.5 m with crest width of 4 m and side slope 1:1.5. It was constructed in three stages as indicated in Fig.18, and the embankment was completed in little over one month. The cross section of the embankment and foundation soil is shown in Fig.19.

The stress applied by the fill was about 27 kN/m^{2}and based on the original shear strength of the clay. The factor of safety against failure was about 1.5, still from elastic theory; the maximum shear stresses under the edge of the embankment were quite high and may have locally exceeded the shear strength.

Instrumentation was installed near the middle of the embankment at locations shown in Fig. 19. Two types of gages were used to measure settlements. Those were placed on the ground surface under the fill where simply steel rods welded to plates with settlement recording made by precise leveling. The gages for measuring settlements at depth considered of rod to firm bottom width in pipe welded to earth screw. Dial indicators determined relative movements. Horizontal movements at the sides of the embankment were measured in plastic tubes by the (SGI) inclinometer (Kallstenius And Berqau, 1961).

Soil Properties

The geotechnical profile for the soil under embankment is shown in Fig. 20 (Holtz, 1972). The soft soil was divided into 8 sub-layers with different compressibility parameters and (OCR) (Neher et al., 1999) as shown in Table 2.

Table (2) Soil parameters for Skä Edbey embankment (from Neher et al., 1999).

<table>
<tr><th>Layer</th><th>γ_t (kN/m^3)</th><th>k (m/day)</th><th>λ</th><th>OCR</th><th>M</th><th>ϕ</th></tr>
<tr><td>A</td><td>14.2</td><td rowspan="2">8.64*10^{-5}</td><td>0.106</td><td>14.1</td><td rowspan="8">1.61</td><td rowspan="8">30°</td></tr>
<tr><td>B</td><td rowspan="2">14.4</td><td>0.096</td><td>2</td></tr>
<tr><td>C</td><td>7.0*10^{-5}</td><td>0.083</td><td>1.2</td></tr>
<tr><td>D</td><td rowspan="5">16.1</td><td>6.22*10^{-5}</td><td rowspan="4">0.076</td><td rowspan="5">1.0</td></tr>
<tr><td>E</td><td>5.44*10^{-5}</td></tr>
<tr><td>F</td><td>5.01*10^{-5}</td></tr>
<tr><td>G</td><td rowspan="2">4.75*10^{-5}</td></tr>
<tr><td>H</td><td>0.069</td></tr>
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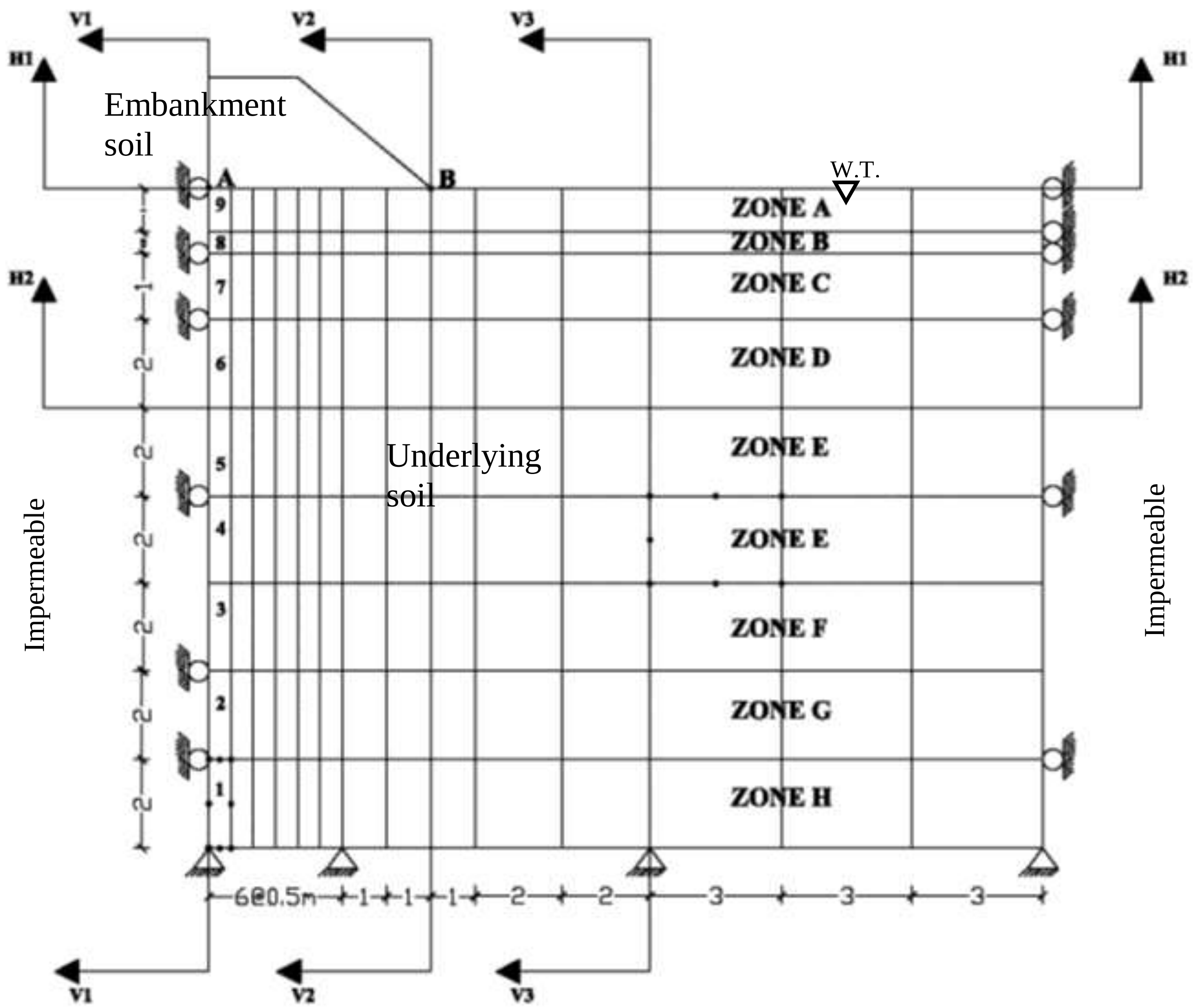
Comparison of Results with Observation: -

The predicted and measured vertical settlement at node A (as shown in Fig. 20), and excess pore water pressure at section (V1-V1) at time 10 years after construction and horizontal displacements along section (V2-V2) at time 20 years are shown in Figs. (23, 24 and 25).

It can be seen that the results obtained using the CRISP program are in good agreement with those obtained by PLAXIS program by adopting the modified Cam clay model. The accuracy of models used and the correctness of the input property of the soil, typically for soft ground, is a big topic of soil mechanics, since the elasto-plastic, visco-plastic, and reheology behavior of the soft clay make it extremely difficult to model the soft soil stress- strain relationship. Normally, increasing the model parameters may increase the accuracy of the model, but also increase the difficulties for input parameters evaluation. At present, some of the model parameters, such as compressibility, can be determined from high quality laboratory triaxial or consolidation tests with confidence. However, some other parameters, such as permeability, are very difficult to be determined accurately. For this kind of parameters, the most reliable way is to derive the parameter values from back- analysis of existing case histories.



V1
V2
V3
H1
H1
Embankment soil
A
B
W.T.
ZONE A
ZONE B
ZONE C
ZONE D
ZONE E
ZONE E
ZONE F
ZONE G
ZONE H
H2
H2
Underlying soil
Impermeable
Impermeable
9
8
7
6
5
4
3
2
1
6@0.5m
V1
V2
V3

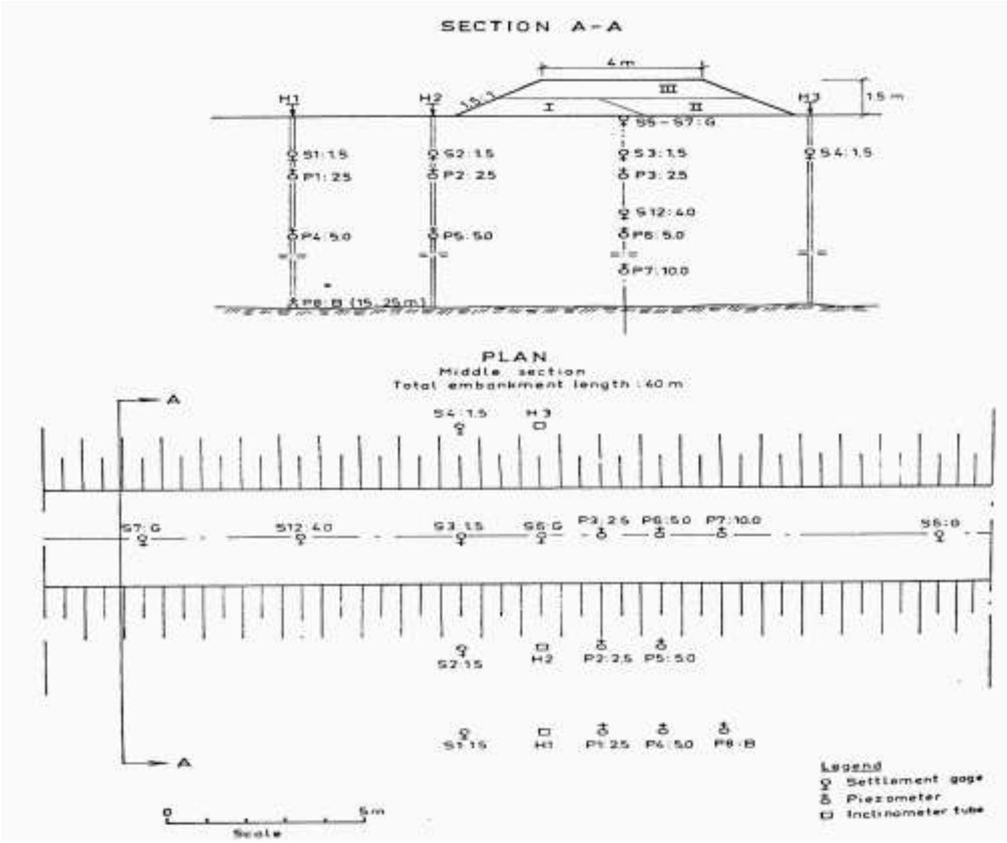


Fig. 21 Plan and section of Skä Edbey embankment showing gage locations and depth below ground surface in meters) (from Holtz, 1972).

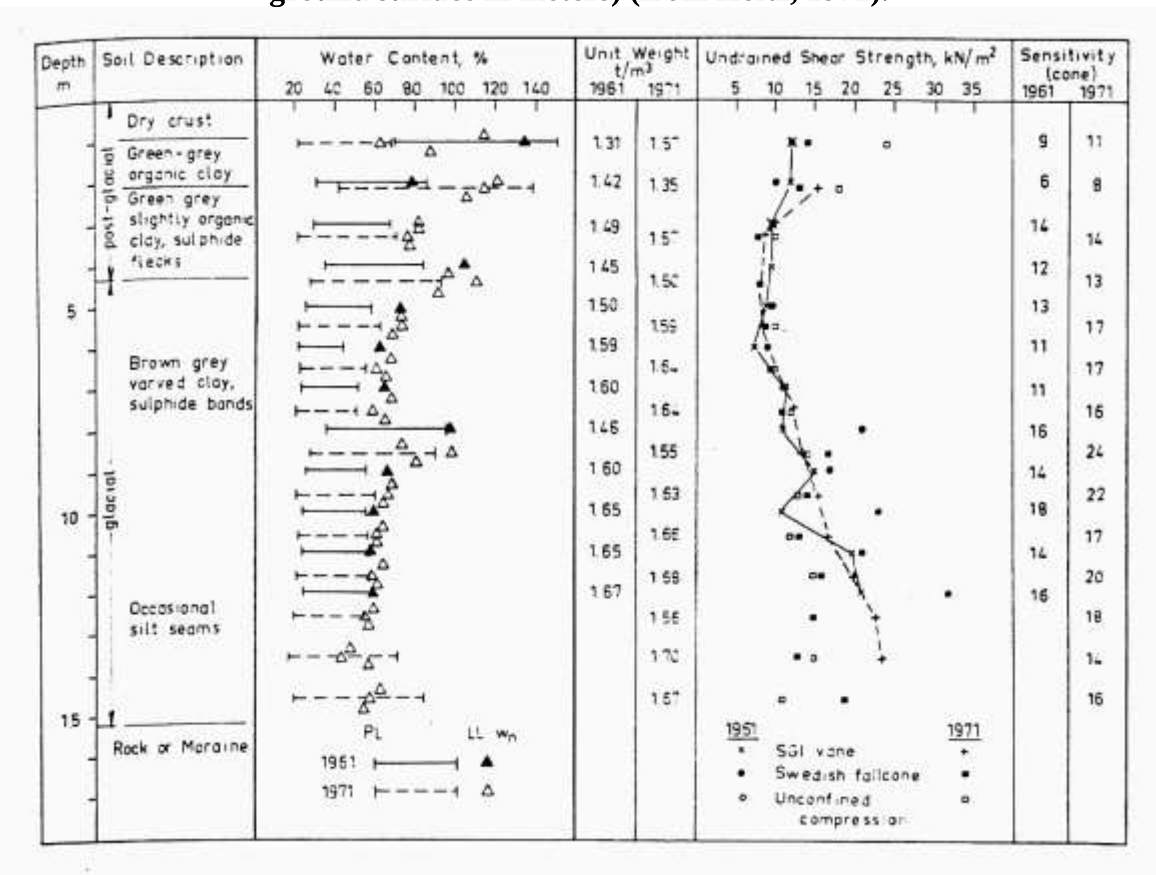


Fig. 22 Geotechnical profile for Säk Edbey embankment (from Holtz, 1972).

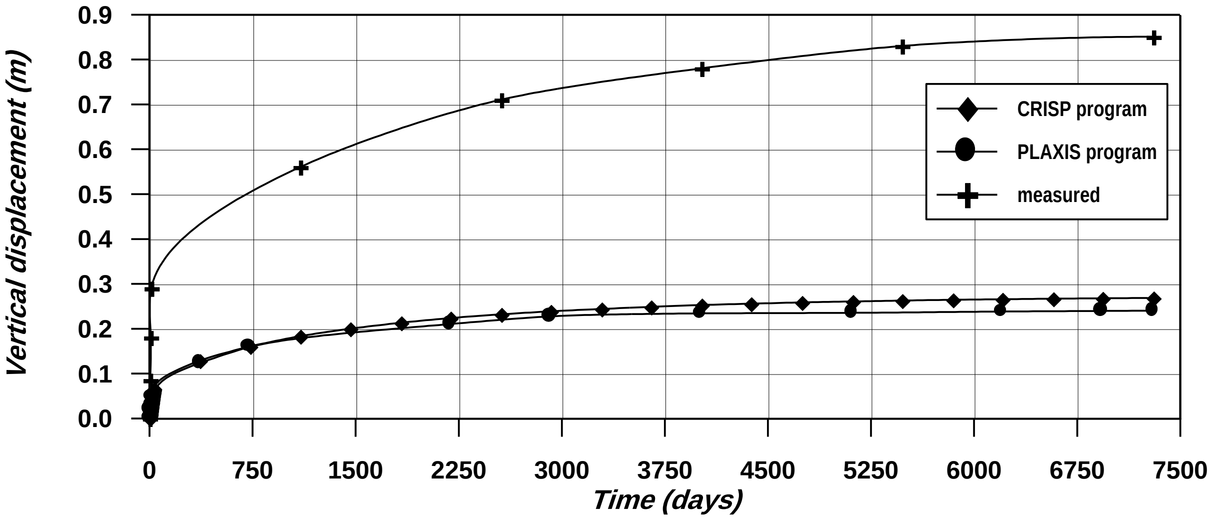


Fig.23 Vertical displacement at point (A) at 20 years after construction of Skä Edbey test embankment.

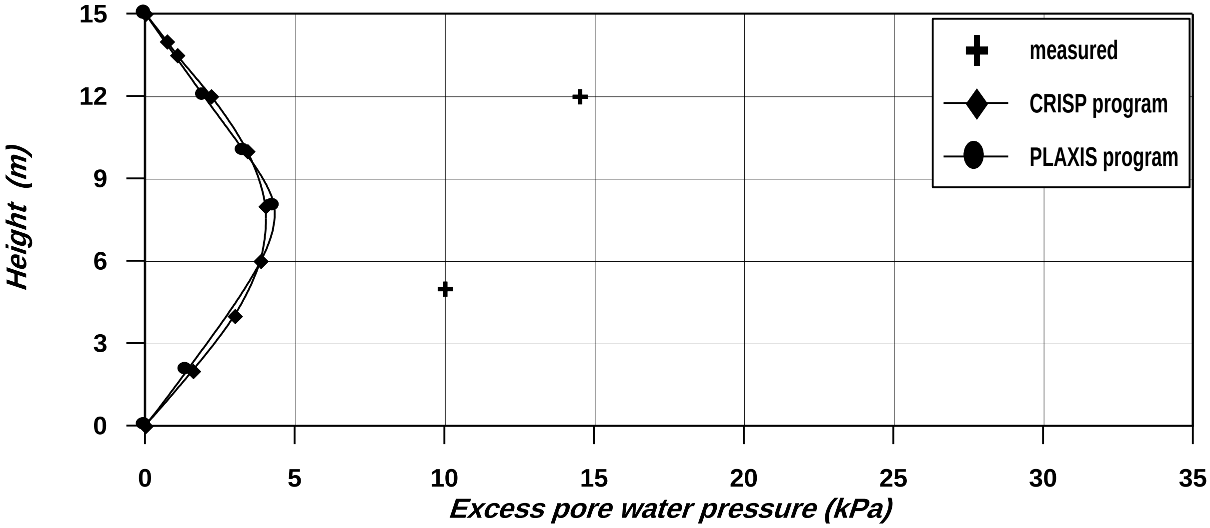


Fig. 24 Excess pore water pressure along section (V1-V1) at 10 years after construction of Skä Edbey test embankment.

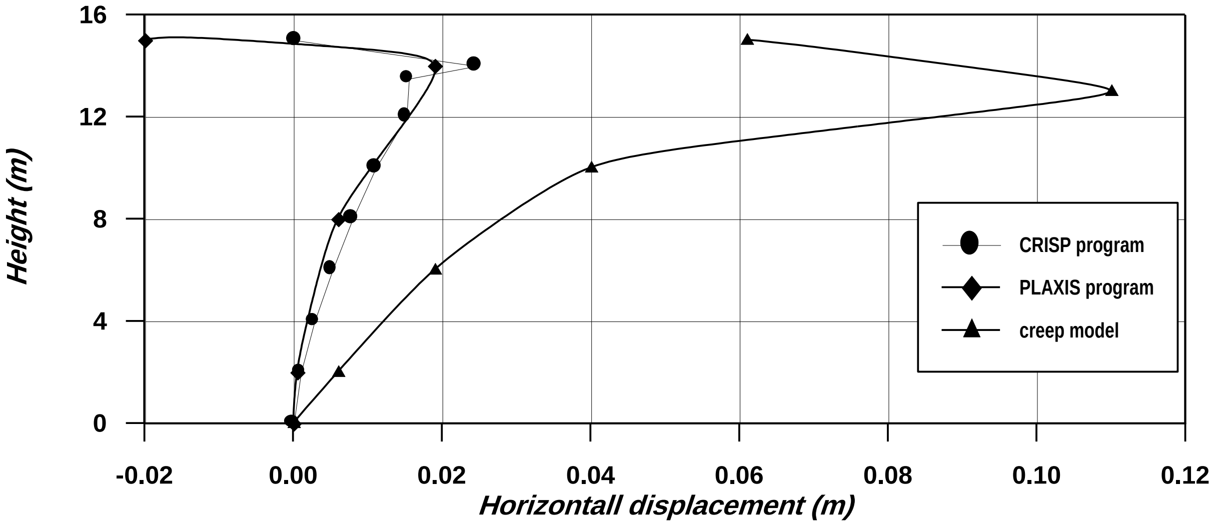


Fig. 25 Horizontal displacement along section (V2-V2) at 20 years after construction of Skä Edbey test embankment.

-Vertical Settlement

For the Skä Edbey test embankment, the vertical movements at the ground surface, sections (H1-H1) and (H2-H2) (shown in Fig. 20) are presented in Figs. (26 and 27). As can be seen, the maximum vertical settlement occurred below the centerline of the embankment. The settlement decreases slightly as the toe of the embankment is approached and decreases rapidly as the distance away from the toe increases. Upward movement of the surface far from the toe is observed. This behavior is due to the loading concentration at the center part of the embankment.

Vertical displacements along sections (V1-V1), (V2-V2) and (V3-V3) are shown in Figs. (28, 29 and 30).

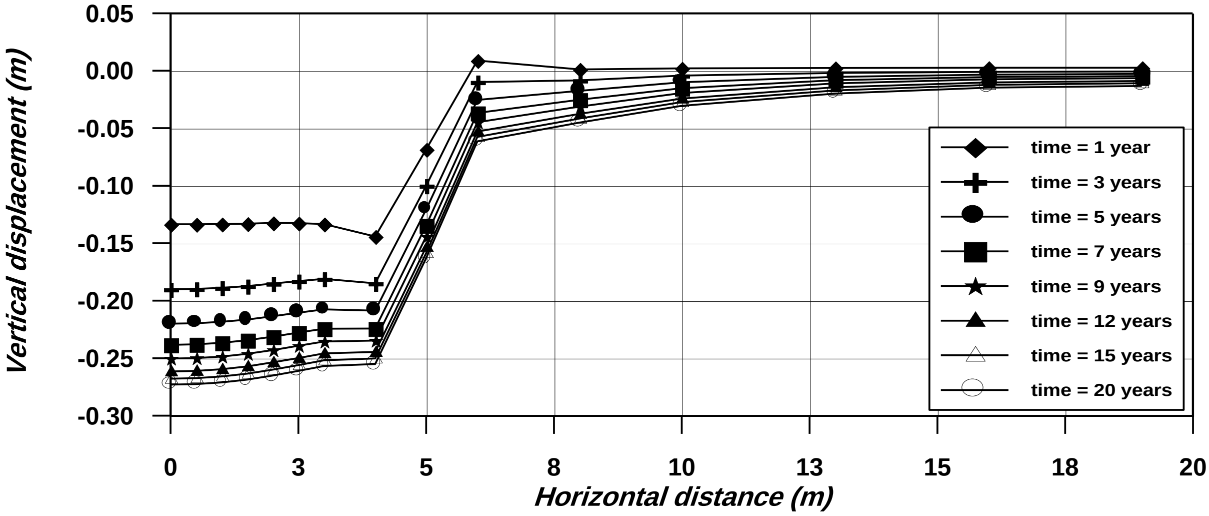


Fig. 26 Vertical displacement along section (H1-H1) after construction of Skä Edbey test embankment.

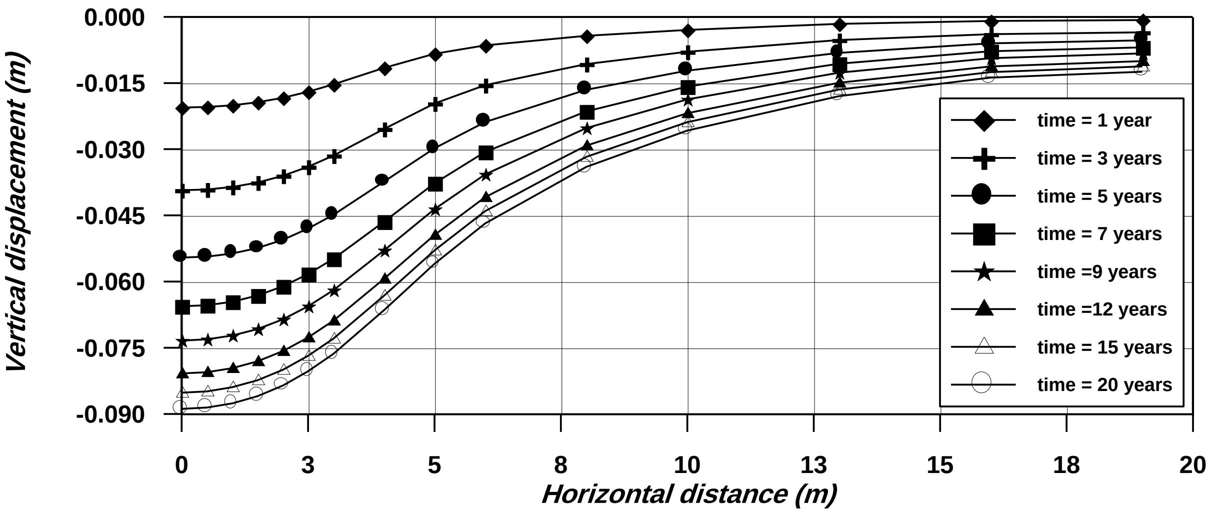


Fig. 27 Vertical displacement along section (H2-H2) after construction of Skä Edbey test embankment.

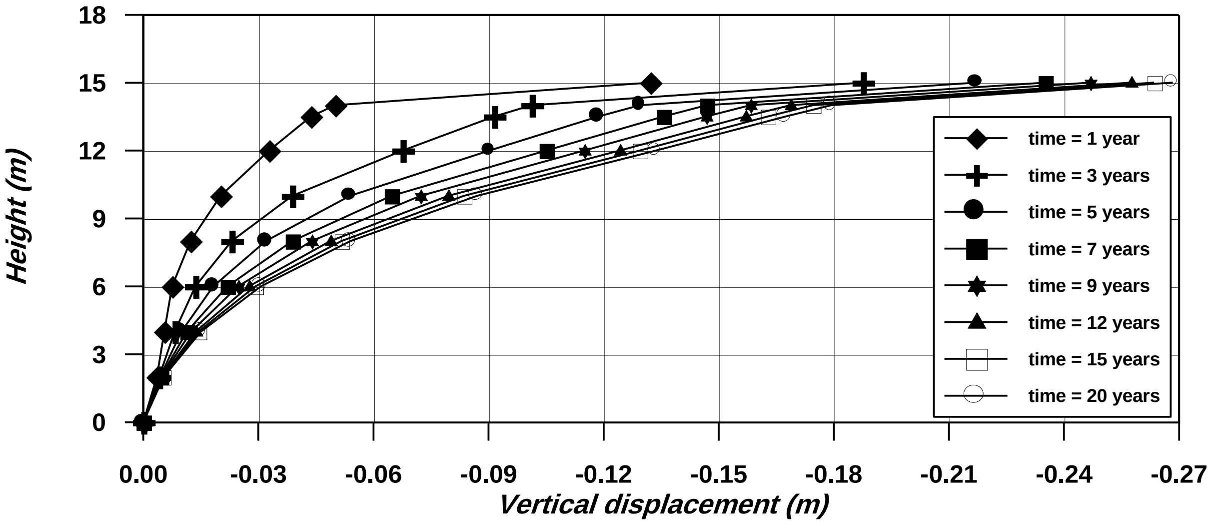


Fig. 28 Vertical displacement along section (V1-V1) after construction of Skä Edbey test embankment.

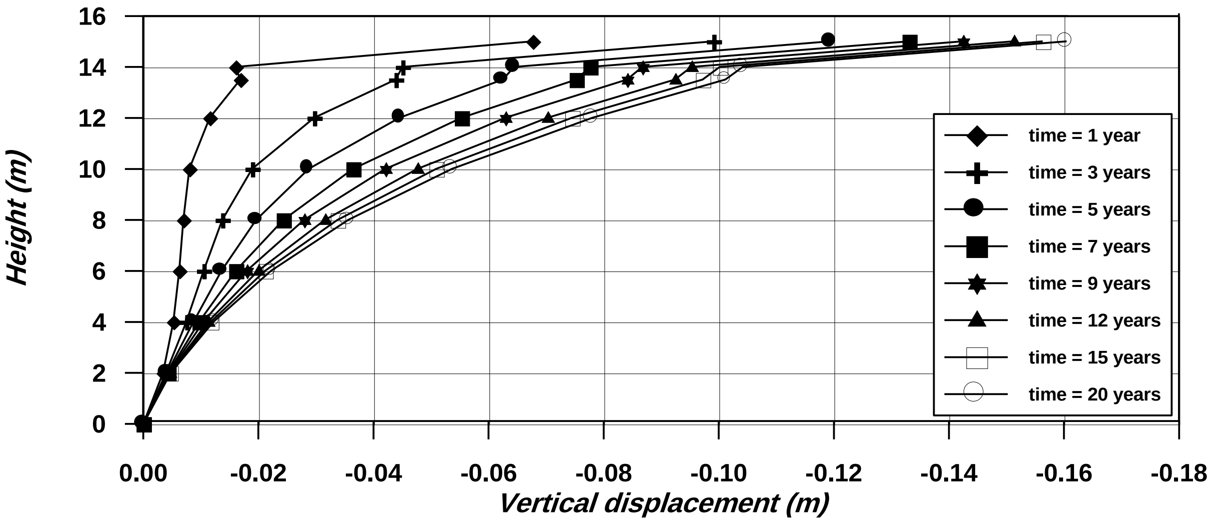


Fig. 29 Vertical displacement along section (V2-V2) after construction of Skä Edbey test embankment.

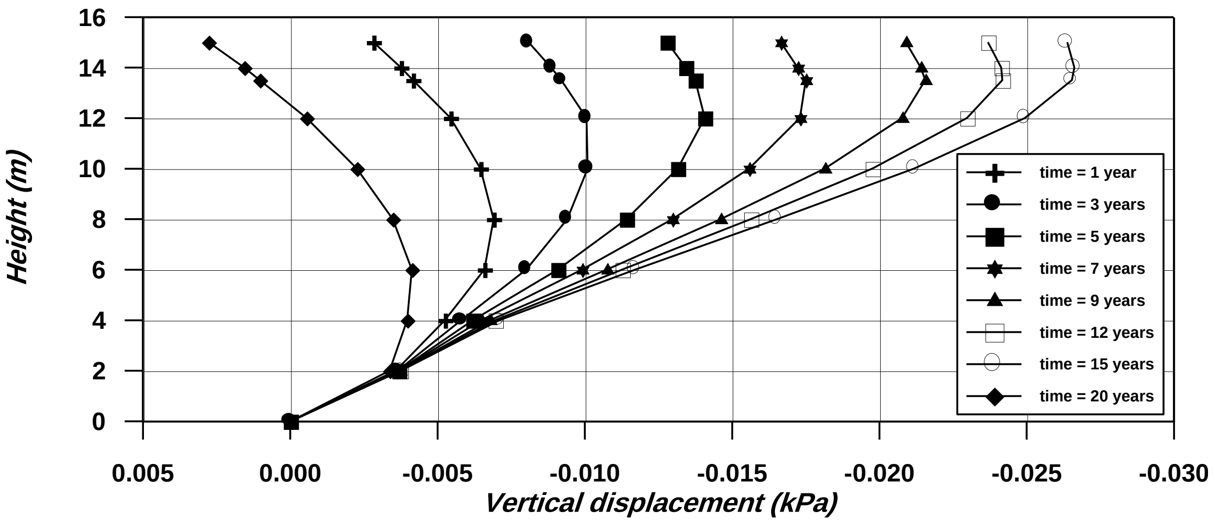


Fig. 30 Vertical displacement along section (V3-V3) after construction of Skä Edbey test embankment.

Horizontal Displacement

The horizontal movement along two vertical sections, which are (V2-V2) and (V3-V3) shown in Fig. 20 are presented in Figs. (31 and 32) after construction at different times. In general, it can be seen that the maximum horizontal displacement occurs near the top boundary. The rate of horizontal movement at top of the foundation is greater than that at the bottom. This behavior may be due to the flexibility of the top boundary that is assumed for horizontal movement. It can also be seen that section (V2-V2) at the toe, shows the maximum horizontal displacement compared with the other section. This behavior is due to the large difference between the vertical load on the left and right hand sides of section (V2-V2).

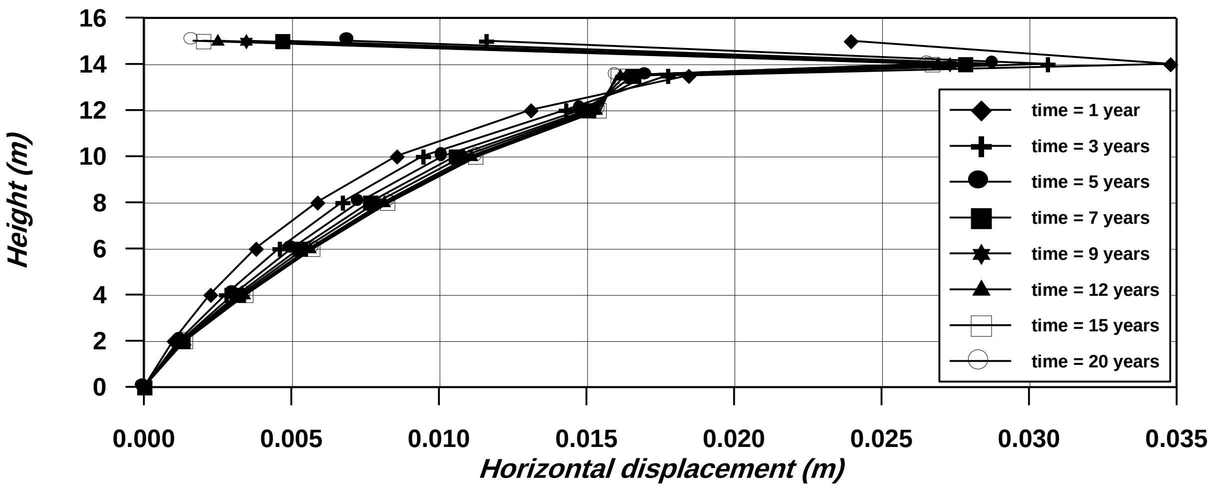


Fig. 31 Horizontal displacement along section (V2-V2) after construction of Skä Edbey test embankment.

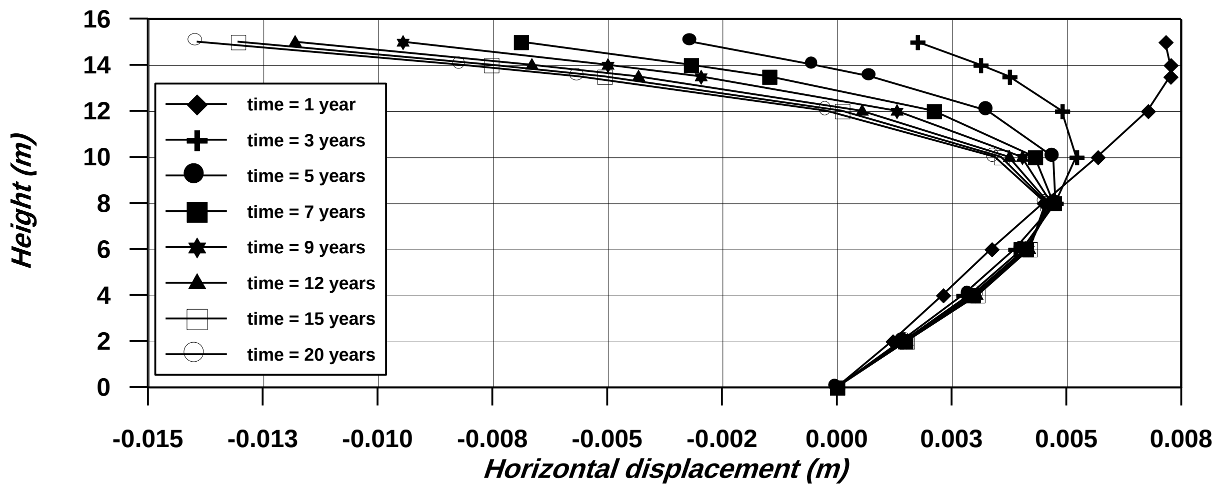


Fig. 32 Horizontal displacement along section (V3-V3) after construction of Skä Edbey test embankment.

Pore Water Pressure

The excess pore water pressure dissipation with time at sections (V1-V1), (V2-V2) and (V3-V3) is shown in Figs. (33, 34 and 35).

As shown in the figures, there is no symmetry in excess pore water pressure isochrones in spite of tow- way drainage condition. This may be attributed to the non-homogeneity in the foundation soil and the differences in the permeability values for different types of foundation layers. The dissipation of excess pore water pressure in the upper layer was faster than at the bottom layer. This is because that the upper layer has permeability greater than that of the underling layers.

The excess pore water pressure along section (V1-V1) is higher than of the two other sections (V2-V2) and (V3-V3). Such behavior is clearly due to the location of the embankment loads, which are more concentrated on the centerline of the embankment.

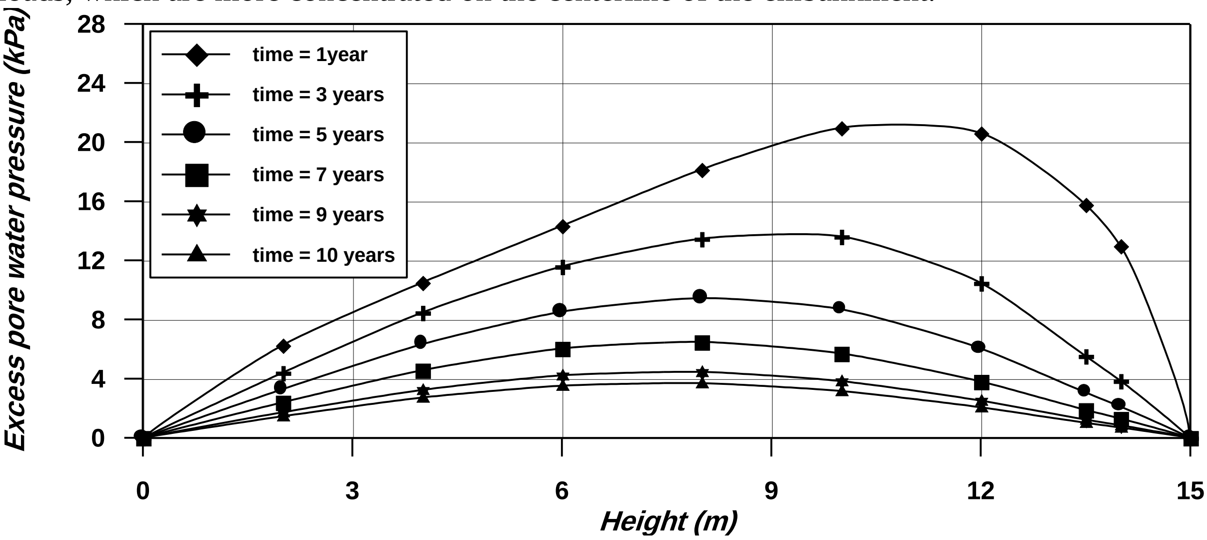


Fig. 33 Excess pore water pressure along section (V1-V1) after construction of Skä Edbey test embankment.

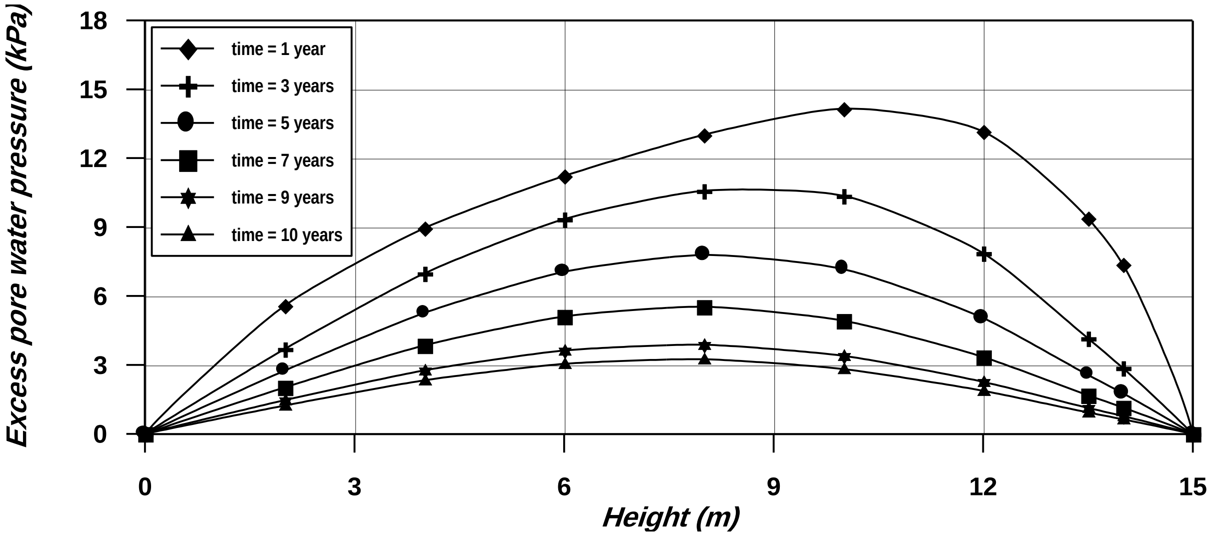


Fig. 34 Excess pore water pressure along section (V2-V2) after construction of Skä Edbey test embankment.

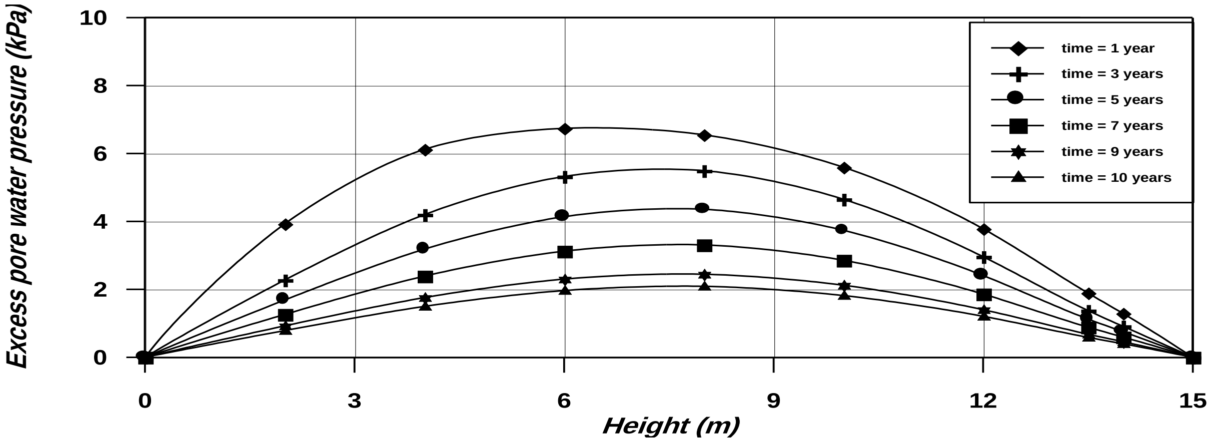


Fig. 35 Excess pore water pressure along section (V3-V3) after construction of Skä Edbey test

embankment.

-Stress State

Figs. (36 and 37) trace the stress path for points A and B (shown in Fig. 20), respectively. It can be seen that isotropic hardening takes place upon loading and new yield surface is always generated at each load increment. The hardening at point (A) which is located at the centerline of the embankment is greater than that at point (B) at its toe. In addition, due to the upward movement of point (B), the stress state may over-pass the critical state line at some stages, and hence dilation might occur at the point. The soil behaves as overconsolidated at that stage.

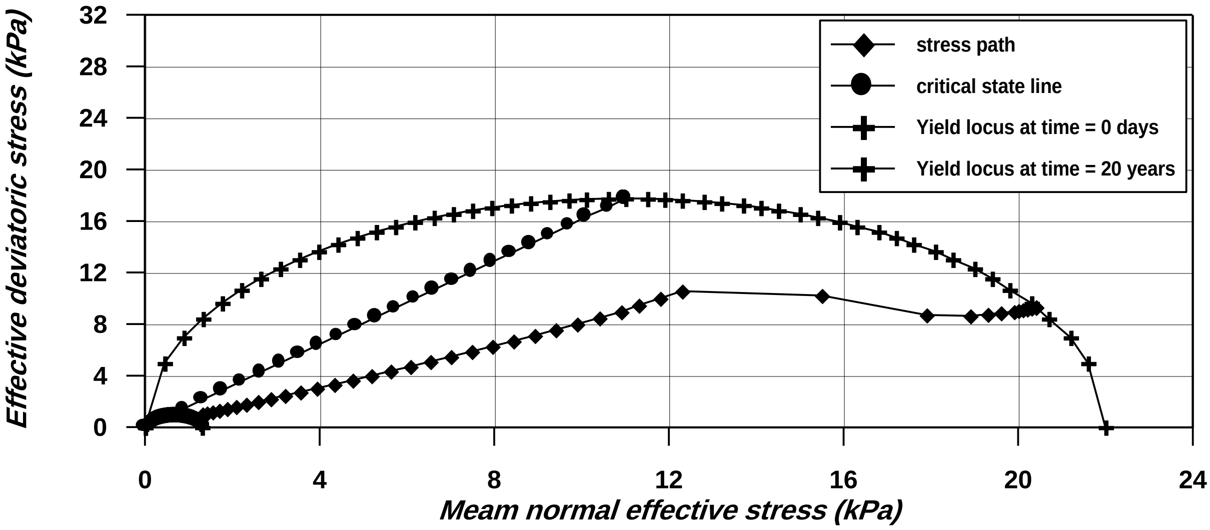


Fig. 36 Stress state at node (A).

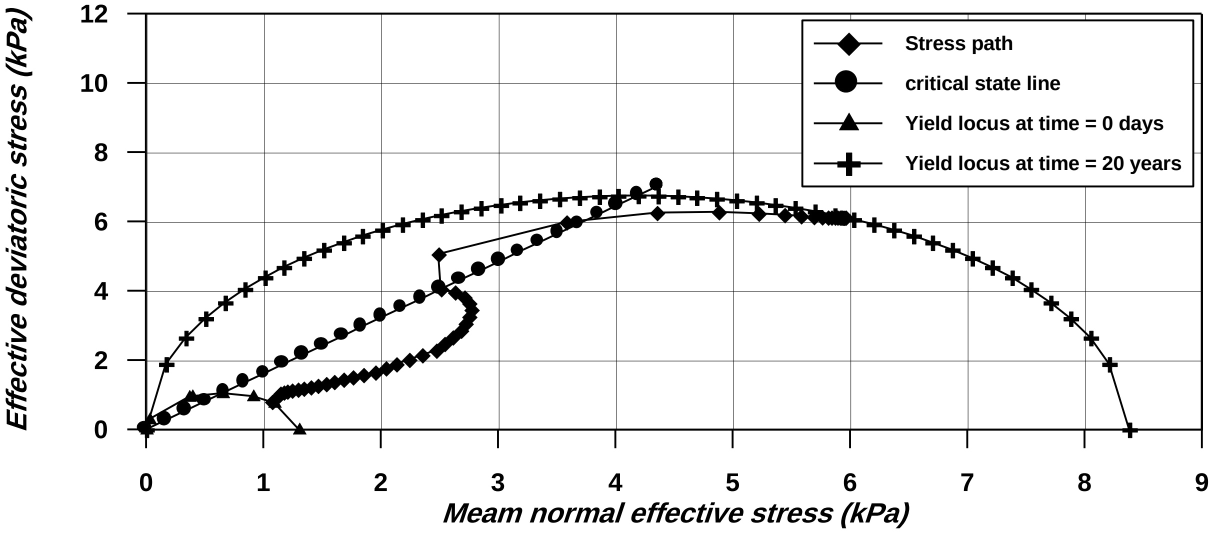


Fig. 37 Stress state at node (B).

CONCLUSIONS

The analysis of two study embankments on soft ground by using the finite element method is achieved. The embankment material is assumed to behave as elastic or elastic-plastic material while foundation soil behavior is considered to follow the modified-Cam clay model. The following conclusions can be drawn:

1. The modified-Cam clay model can simulate the soil behavior successfuly. When the results of analysis are compared with the measured values, good agreement was obtained.

2. The maximum vertical movement occurs below the centerline of the embankment. The settlement decreases slightly as the toe of the embankment is approached and decreases rapidly as the distance away from the toe increases. Upward movement of the surface far from the toe is observed.
3. The maximum horizontal movement occurs near the top boundary. The rate of horizontal movement at the top of the foundation is greater than at the bottom. This behavior may be due to the flexibility and free movement condition of the vertical boundary in the top half.

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