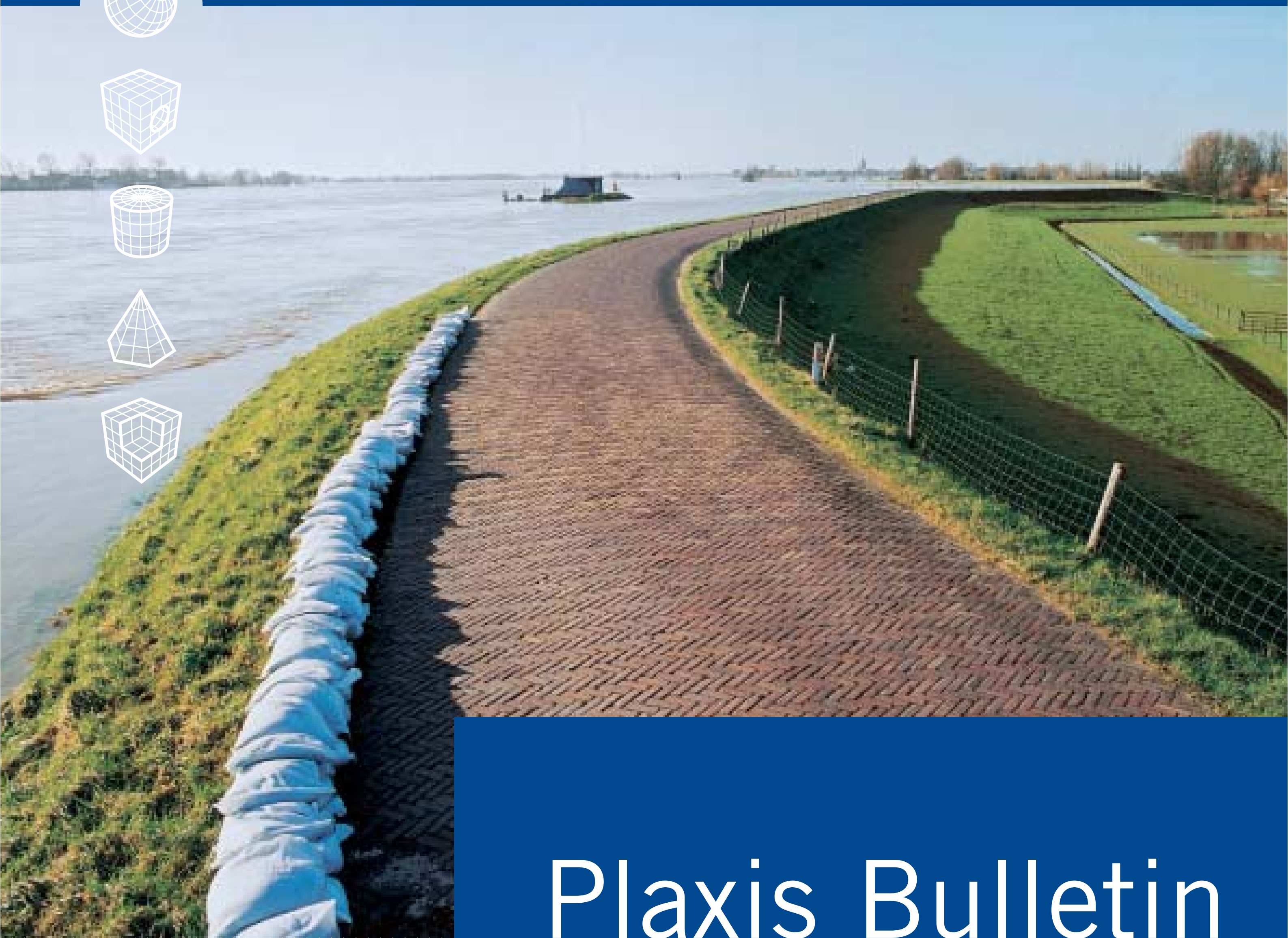
PLAXIS FINITE ELEMENT CODE FOR SOIL AND ROCK ANALYSES



Plaxis Bulletin

issue 14 / september 2003

PLAXFLOW A USEFUL TOOL in an advanced assessment of the 'risk of sliding of an inner slope cover during wave overtopping'

The interaction between Plaxis V8 and PlaxFlow

Load-settlement response of a footing bearing

PLAXIS

Cover photo: Meetkundige Dienst Rijkswaterstaat, Ministerie V&W

LAST JUNE WE PROUDLY RELEASED THE LATEST ADDITION OF THE PLAXIS FAMILY OF PROGRAMS; PLAXFLOW, THE FINITE ELEMENT CODE FOR TRANSIENT AND UNSATURATED GROUND-WATER FLOW. THIS BULLETIN PAYS SPECIAL ATTENTION TO PLAXFLOW AND THE POSSIBILITIES IT OFFERS. THE PROGRAM WAS OFFICIALLY RELEASED DURING A MINI-SYMPOSIUM IN DELFT. UNDER THE HEADER OF RECENT ACTIVITIES YOU WILL FIND A SHORT IMPRESSION OF THIS DAY.

Editorial Staff:
Dr. Ronald Brinkgreve, Dr. Wout Broere, Mr. Marco Hutteman

PlaxFlow is the result of a cooperation between Plaxis and GeoDelft. It allows not only for the calculation of groundwater flow, but in combination with Plaxis V8, also for the analysis of the interaction between groundwater flow and deformation analysis. The strength of such a tool is shown in one of the Plaxis Practise articles in this issue.

To help the new PlaxFlow user unlock the full power of both PlaxFlow and Plaxis V8, a short introduction to the interaction between the two programs has been written. Intended as an extension to the Tutorial Manual of Plaxis V8, it will quickly familiarize you with the basics of the interaction between time-dependent groundwater flow and deformation analysis. You will find this in the column Plaxis Tutorial.

You will find also other regular columns in this issue, such as New developmens. This issue the upcoming first release of Plaxis 3D Foundation is discussed. This will be the next new addition to the family of Plaxis products and is a fully 3D program, intended for the analysis of foundations.

Unfortunately, in the last issue of the Plaxis bulletin, an incorrect version of the new Benchmark case description was presented. With this description, it was not possible to complete the benchmark as intended. We apologize for the inconvenience. In this issue you will find the correct text for the new benchmark and we hope that many users will still take up the challenge to send in results.

Next month we will organize a course on Computational Geotechnics in cooperation with the Sociedad Mexicana de Mecánica de Suelos in Mexico. And also for the following months the Schedule is filled with courses and user meetings and we hope to welcome you at one of them.

New developments

Author: Dr. Ronald Brinkgreve

Although this Bulletin is devoted to PlaxFlow, I like to address in this column the next product that will be released: Plaxis 3D Foundation. With this product Plaxis really enters into the third dimension! No doubt, the existing 3D Tunnel program is already a true 3D program, but due to the geometrical limitations this is not fully recognized by many users. The 3D Foundation program is the proof that 'User-friendly 3D modelling' is not a contradictory expression. But how does it work?

In contrast to existing Plaxis products, modelling in the 3D Foundation program is done from a top view. Distinction is made between the modelling of the sub-soil and the modelling of structures and loadings.

Regarding the sub-soil, an arbitrary number of 'bore holes' can be defined in the top view. In each bore hole, information about the local soil layer positions and pore pressure distribution must be specified. During 3D mesh generation, this information is interpolated between the bore holes, and this forms the basis of the 3D mesh. The concept allows for an arbitrary non-horizontal soil stratification and ground surface modelling including slopes.

Regarding structural behaviour, an arbitrary number of horizontal 'work planes' can be defined at different levels. Structures (beams, walls, floors, springs, piles) and loads can easily be created in work planes (horizontal) or between work planes (vertical). In the first release, piles are modelled using (curved) volume elements, with interface elements around to model the pile-soil interaction. It is recognized that this concept puts limitations on the number of piles that can be modelled in this way, but new research and development is planned for the next version in which piles are modelled in a more efficient way using special line elements.

The 3D foundation program enables the modelling and analysis of foundation structures in which the interaction between the structure and the sub-soil plays an important role. This is for example the case for ordinary raft foundations, and more pronounced for pile-raft foundations or offshore foundations (suction piles). Due to the easy modelling of non-horizontal soil layers and ground surface, the program is particularly useful to analyse the stability of foundations on slopes. Moreover, the program may be used to overcome the geometric modelling limitations of the current 3D Tunnel program. Hence, the range of applications is larger than just foundations.

The 3D Foundation program will be released in the first quarter of 2004.

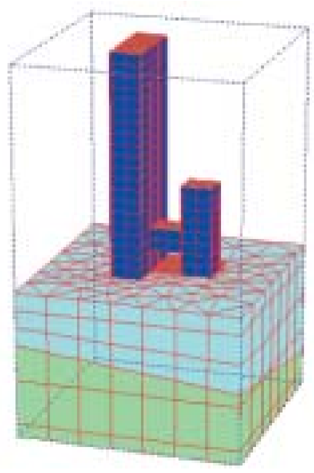


Figure 1: Example of foundation structure

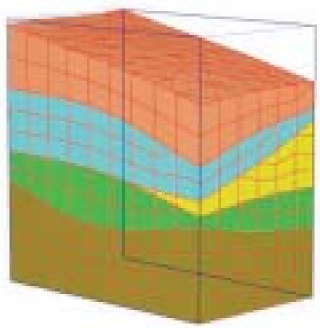


Figure 2: Non-horizontal soil stratification

BENCHMARK NO. 3: EMBANKMENT 1

Author: Helmut F. Schweiger, Graz University of Technology

THE GEOMETRY OF THE EMBANKMENT EXAMPLE FOLLOWS FROM FIGURE 1. A LAYER OF 2 M HEIGHT HAS TO BE CONSTRUCTED. THE WATER TABLE IS ASSUMED TO BE 1.0 M BELOW THE SURFACE. FOR THE FIRST METER OF THE GROUND PROFILE A GRAVEL LAYER IS ASSUMED (TOP LAYER IN FIGURE 1). THE SOFT CLAY UNDERNEATH HAS A THICKNESS OF 29 M AND HAS TO BE MODELLED WITH THE SOFT SOIL MODEL.

GENERAL ASSUMPTIONS:

- plane strain
- boundary conditions for consolidation: lateral and bottom boundaries closed
- stiff strata below soft clay, i.e. displacements restrained at level y=30 m

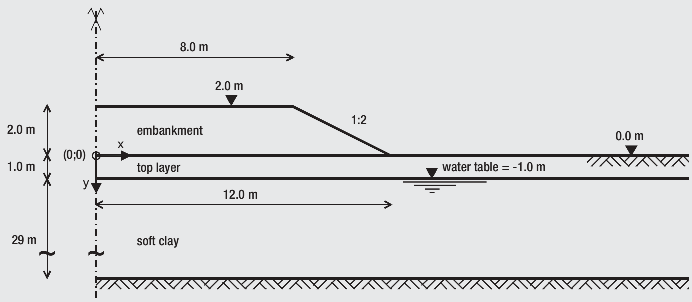


Figure 1: Geometry of embankment benchmark

THE FOLLOWING COMPUTATIONAL STEPS HAVE TO BE PERFORMED:

- initial stresses, with $\sigma'_V = \gamma.y$, σ'_H considering a pre-overburden pressure (POP) of 10 kPa
- apply embankment layer under undrained conditions
- 25 years of consolidation

MATERIAL PARAMETERS

Embankment and top layer: Mohr-Coulomb model

γ	E	φ	ψ	c	ν	Tensile-Strength
kN/m³	kPa	°	°	kPa	-	kPa
20.0	40 000	38	0	1.0	0.3	0.0

Soft clay: Soft Soil model

κ	λ	φ	ψ	c	ν	γ_{sat}	e_0	k_f
-	-	°	°	kPa	-	kN/m³	-	m/s
0.028	0.15	27	0	0.1	0.2	18.5	1.1	$1*10^{-9}$

RESULTS TO BE PRESENTED AFTER CONSTRUCTION OF THE EMBANKMENT AND AFTER CONSOLIDATION, SEPARATELY (ORIGIN OF COORDINATE AXES FOLLOWS FROM FIGURE 1):

- time settlement curve for point x = 0, y = 0
- surface settlement trough
- horizontal displacements at vertical profile at x = 12 m
- effective stresses σ'_x and τ_{xy} at vertical profile at x = 12 m
- effective stresses σ'_x, σ'_y and τ_{xy} at horizontal profile at y = 2.5 m

RESULTS TO BE PRESENTED AFTER CONSTRUCTION ONLY:

- excess pore pressures at vertical profiles at x = 0 m and x = 12 m
- excess pore pressures at horizontal profile at y = 2.5 m

STRESS PATHS:

- excess pore pressure vs time and p'-q-stress path at the following locations (approximately):

  x = 0 > y = 1.5 and 10 m
  x = 12 > y = 1.5 and 10 m

USERS ARE KINDLY REQUESTED TO ANALYSE THIS SITUATION AND TO SEND IN THE RESULTS. ALL RESULTS HAVE TO BE PRESENTED IN EXCEL-SHEETS (PLAXIS-PROJECT FILES OPTIONAL) AND MAILED TO: HELMUT.SCHWEIGER@TUGRAZ.AT

THE RESULTS OF THIS BENCHMARK WILL BE PUBLISHED IN THE NEXT BULLETIN.

DEADLINE: 15 NOVEMBER 2003

RECENT ACTIVITIES

NEW PLAXIS STAFF

Mr Erwin H.A. Beernink MSc. has joined the PLAXIS company as the new head of Marketing and Sales. He took over from Dr Wout Broere who has filled this position as Interim Manager after that Mr Peter Brand left PLAXIS BV. Dr Broere will continue his work on courses and support. In his new position Mr. Beernink will be responsible for the continued growth and direction of the company's marketing and sales. He has been involved with marketing and business development in the civil engineering area for more than 15 years.

Furthermore the Plaxis staff has been expanded with a scientific employee, Dr. Erick Septanika and a GUI programmer Mr. Ernst Dunnink.

Scientific co-worker Erick is graduated from the Faculty of Aerospace Engineering of TU Delft and has a doctor degree on Computational Mechanics from the Faculty of Mechanical Engineering TU Delft. Before he jointed Plaxis, he worked for MARC and TNO-DIANA finite element software companies. In Plaxis, he is working on the development and implementation of new material models and new techniques on failure mechanism. Further, he is also involved in the development of the Plaxis 3D Foundation Specials. Erick plays the bass guitar and listens to jazz music.

Ernst Dunnink studied Business Information Technology and graduated at the University of Twente. Specializations during his studies were system development and knowledge technology. After graduation he worked for a small engineering company in Gouda which produces computer programs based on calculation models. Within Plaxis he will perform the role of user interface programmer.

COME BACK DEMO CD

Due to increasing number of activities we had a little delay in launching the Introductory version of Plaxis V8. Now we are proud to announce the come back of this free Demo version of our general 2D geotechnical FE product. Furthermore we released Plaxis Version 8.2, latest update of 3D Tunnel and PlaxFlow.

Furthermore the Reference manual of Version 8 is available in German, French and Italian.

PLAXFLOW INTRODUCTION

On June 5th the easy-to-use groundwater flow analysis tool was officially launched. PlaxFlow is a FEM code with the most appealing capabilities in the field of steady state and transient groundwater flow for saturated and unsaturated conditions. The program is specially designed for use by geotechnical engineers and offers a very user friendly Graphical User Interface for both problem definition and parameter input.

Presentations were held both on the easy to use interface and the theoretical background. The easy-to-use geometry input allows for the modeling of screens, wells and drains. Furthermore, boundary conditions can be defined in terms of pore pressures, heads, external water pressures and/or precipitation. All of these can be constant or variable in time. Material models can be defined with several levels of sophistication. Soil hydraulic properties in PlaxFlow can be described by a comprehensive set of predefined data sets of well-known international soil classes or by the parametric functions of Van Genuchten. The Van Genuchten formulation utilizes a three-phase model for saturation-pressure - permeability relations. Besides predefined sets and direct input of the Van Genuchten parameters the user can also apply user-defined saturation-pressure -permeability relations. All material models allow for the modeling of anisotropic flow conditions.

PlaxFlow can be used as a stand-alone program or as an integrated module of Plaxis V8. Plaxis Version 8.2 in combination with PlaxFlow Version 1 offers the possibility to take the influence of complex transient flow conditions on soil deformation and stability into account.

AGENT FOR AUSTRALIA

Mr. Nicolas Vass

From September 2003 Plaxis appointed a new agent for Australia. Any inquiries can be directed to the Techsoft Australasia Pty Ltd office in Brisbane. Address is 5 Cameron St. Beenleigh Queensland 4127 Australia. The fax number is +617 3807 8422, email: plaxis@techsoft.com.au.

Managing Director of the company Mr. Nicolas Vass has a solid background in business development and product sales/marketing management. Techsoft Australasia is attuned to dealing with international vendors and suppliers from Europe, US, Australia, Japan & Taiwan. Nicolas has a long term experiences with direct sales and support of Ove Arup Structural & Geotechnical , MSC Structural Analysis FEM based Software, PLAXIS Geotechnical Software and other computer aided-engineering applications for civil/mechanical/structural/geotechnical applications.

COURSES

The 1st Asian Experienced Plaxis Users Course was conducted prior to 12th Asian Regional Conference. The course was held in Singapore and was hosted by A/Prof. Tan and his staff members of the National University of Singapore (NUS). If you did not have a chance to attend this course you can check the agenda for upcoming courses and other events.

Participants of the 1st Asian Experienced Plaxis Users Course

PLAXFLOW AS A USEFUL TOOL IN AN ADVANCED ASSESSMENT OF THE 'RISK OF SLIDING OF AN INNER SLOPE COVER DURING WAVE OVERTOPPING: IJSSELMEERDIJK'

Authors: Dr. Stan Schoofs, Drs. Ing. Remco Schrijver, Ing. Alexander van Duinen, GeoDelft, Netherlands

INTRODUCTION

This article discusses an advanced assessment for the failure mechanism of wave overtopping. The assessment was carried out for a section of 8.5 kilometres of the water-retaining soil structure 'IJsselmeerdijk' above Lelystad, the Netherlands, by order of the Ministry of Public Works and Water Management, directorate 'IJsselmeergebied' [1] (see Figure 1). During a critical storm on the lake, which statistically will occur once in every 4000 years, this structure experiences an overtopping situation with an average flow rate of 0.5 l/s per running meter. Previously, standard assessment rules led to a negative advice for risk of failure for the mechanism of 'sliding of the inner slope cover during overtopping' for this structure. The slope did pass the exam for the 'erosion' failure mechanism. In the first section, the advanced assessment for sliding of the inner slope cover during overtopping is defined. The succeeding sections describe the assessment execution, with focus on the use of PlaxFlow, a new module to simulate saturated and unsaturated groundwater flow.

Figure 1: IJsselmeerdijk, including the experimental setup on top of the crown of the waterretaining structure. The inner slope is to the right hand side, the fence on the right of the picture is placed on the intermediate shoulder.

ADVANCED ASSESSMENT: 'RISK OF SLIDING OF AN INNER SLOPE COVER DURING OVERTOPPING'

The advanced assessment is performed by the execution of the following steps:

- inspection by a specialist of the inner slope cover, including site determination for a real-scale field test
- soil analysis on the field test site (determination of flow and strength parameters)
- execution of the field test (flow rate 1.0 l/s per running meter)
- calibration Plaxis model to the field test results
- extrapolation of results to critical storm and for the complete section by means of variation calculations

SPECIALISTIC INSPECTION / GEOMETRY

The inner slope consists of an upper and lower slope interrupted by an intermediate shoulder that includes a paved road construction. Only the upper slope needed to be tested for the mechanism of sliding of the slope cover during overtopping. The results of a set of auger borings and the analysis of geological profile holes (holes of approximately 0.75 × 0.75 m^2, from which information such as stratification, soil-type, soil-structure, can be derived) were used to schematise the cover of the upper part of the inner slope (see also Figure 2). The upper cover, half a meter thick, is made of silty clay. The lower cover is also half a meter thick, and made of boulder clay with a sand percentage of more than 50 percents. The inner part of the soil structure consists of sand. In certain environments, an impermeable zone develops at the interface between a clay and sand layer, as a result of chemical leaching from the clay. The results of the site investigation show, that there is no reason to assume the structural presence of such an impermeable zone between the lower cover and the sand. Finally, variations in the material properties of the cover and the cover-thickness over the assessment section are small.

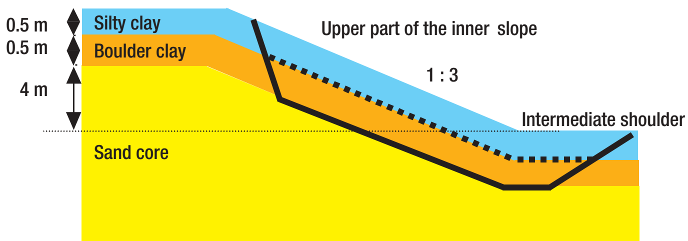


Figure 2: Schematisation of the water retaining structure with focus on the inner slope. The solid and dotted curves show potential sliding surfaces.

DETERMINATION OF (UN)SATURATED FLOW PARAMETERS

In the field, the saturated hydraulic conductivity was determined by performing falling head tests with a double ring infiltrometer (inner ring: ∅ 0.30 m; outer ring: ∅ 0.60 m) at several depths. In the laboratory, the saturated conductivity and the unsaturated material behaviour (Van Genuchten relations [3]) for the two cover materials were determined (see Figure 3). Table 1 shows the average estimates for the saturated hydraulic conductivities of the cover soils, as were measured in several manners.

For use in the PlaxFlow module, the material curves were simplified by assuming a linear relation between moisture content and pressure head and a log-linear relation between conductivity and pressure head (Figure 3). The simplification was set up in a way that the infiltration rate is equal to the rate obtained with the Van Genuchten curves, at least for vertical infiltration in a one-dimensional domain [4]. The use of the adapted curves is a standard option in PlaxFlow. The employed input parameters for the most important materials are shown in Table 2.

Test method	$Ks_{upper\ cover}$ [$\times 10^{-5}$ m/s]	Number of tests	$Ks_{lower\ cover}$ [$\times 10^{-5}$ m/s]	Number of tests
Specialistic guess	10	-	1 to 5	-
Infiltrometer	4	3	2	3
Laboratory	35	2	4	2
In-situ (by flow rate regulation)	-	-	2	1

Table 1: (Averaged) saturated conductivity Ks of the upper and lower cover, including the number of tests.

Material	Ks [m/day]	e (void ratio) [-]	P_s [m]	P_k [m]
Upper cover (silty clay)	17.3	0.67	3.04	1.50
Lower cover (boulder clay)	1.73	0.54	6.58	1.10
Core of the dike structure (sand)	17.3	0.75	2.37	1.06

Table 2: Input parameters for the employed linearised material curves to describe the saturated and unsaturated hydrological behaviour of the most important materials.

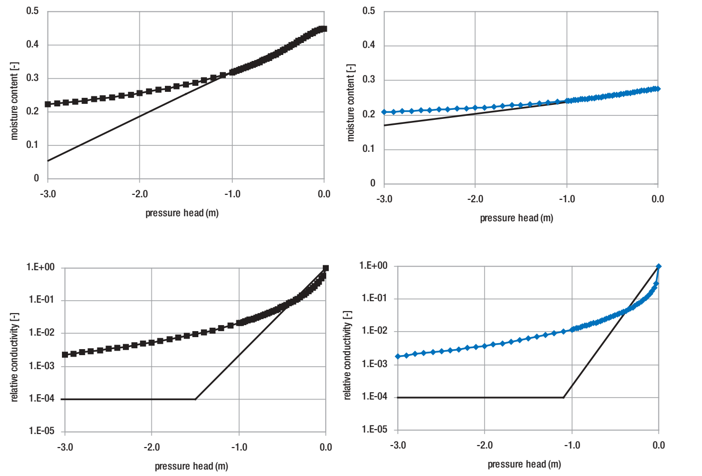


Figure 3: Moisture content θ [-] and relative conductivity $K^r = K / Ks$ [-] as a function of the pressure head (in m), of the silty clay upper cover (left panels) and the boulder clay lower cover (right panels). The curves with datapoints (the points do not represent the measure points) are defined with the Van Genuchten model. The (log-)linearized curves, which were derived from these Van Genuchten curves, were used in the calibration calculations.

DETERMINATION OF STRENGTH PARAMETERS

The strength of the cover material was tested in the laboratory by performing drained simple shear tests on consolidated core samples at low pressures (5, 10 and 15 kPa). The samples were extracted from drill cores. The moisture contents of the samples were varied between natural and complete saturation. The samples were too small to determine the strength of the macro structure, but the specialistic inspection revealed that failure by sliding of the cover is primarily determined by the microstructure. The cohesion and the angle of internal friction of the boulder clay showed a strong dependence on the saturation. Table 3 shows the strength parameters after a safety factor of 1.4 was included. This safety factor represents the measure error, model uncertainty, and the possible soil variations over the assessment section.

The hydraulic properties of the other soil layers were defined on the basis of experience.

In order to test the strength of the interface between the sand and the boulder clay cover, direct shear tests under similar lab-conditions were performed. The results showed that the interface would not be a critical failure surface in comparison with the strength of the boulder clay.

Material	c [kPa]	ϕ [°]	γ_{dry} [kN/m³]	γ_{dry} [kN/m³]	E [kN/m³]	ν [-]
Upper cover	2	21	16	18	30	0.30
Lower cover	3	36	18	21	100	0.33
Sand core	0.001	30	17	20	10000	0.30

Table 3: Employed parameters for cohesion c, angle of internal friction ϕ of the upper and lower covers of the upper inner slope, as used in the calculations (including an uncertainty factor of 1.4). Other parameters given are the wet and dry weight γ, Young's modulus E, and the Poisson 's constant ν.

FIELD TEST

The purpose of the field test was to test the actual slope strength during wave overtopping with a critical flow rate. Complete failure of the slope was undesired.

Prior to the field test the cover was saturated during a period of three days, in order to simulate the initial moisture content during a critical storm (the field test was in spring, critical storm is expected in late autumn or winter). The field test represents the wave overtopping during a critical storm, which is calculated to last a time period of maximal 12 hours. During this time period of 12 hours, a constant flow rate of 1.0 l/s per running meter was imposed on the structure over a length of 30 meters (see Figure 1 and the close up in Figure 4). In reality, the overtopping flow rate will be irregular. However, for the failure mechanism of 'sliding of the cover due to infiltration' a constant, average flow rate is considered to be more critical. Instrumentation consisted of 12 tensiometers to measure pore water pressure. The cups were placed both in the slope cover and in the sandy inner part of the soil structure. Further, three inclinometers were used to measure the deformation of the slope cover. Finally, a set of five points was fixed at the toe of the upper slope in order to measure the vertical deformation.

Figure 4: Close-up of the experimental setup for generating the constant 'overtopping' flow rate of 1 l/s over a stretch of thirty meters

During the initial few hours of the field-test, a substantial amount of air bubbles were observed between the stones of the road at the intermediate shoulder. After some time these bubbles disappeared. This event suggested the saturation of the inner part of the soil structure, at least to the height of the intermediate shoulder.

In the upper panel of Figure 5 the measure deformation of the slope during the field test is shown. The deformation is of the order of 2 to 4 millimetres. One of the curves shows an upward movement along the slope (of the recorder which is located in the middle of the slope cover), while the other two curves (recorders located in the lower part of the slope) show deformation in a downward direction. The reason of the difference in deformation direction is not clear. A possible explanation would be the swelling of the slope cover due to the water infiltration.

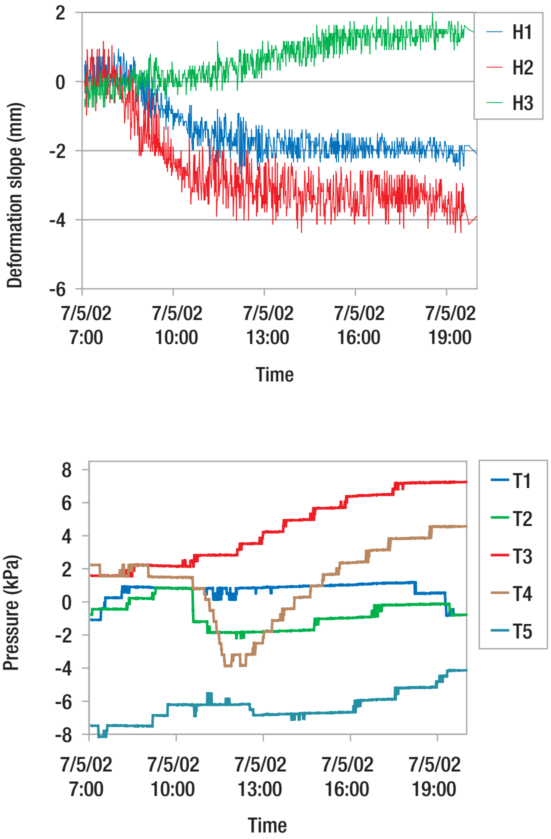


Figure 5: Measured deformation of the inner slope in millimeters along the slope (top) and measured pressures at five locations in the boulder clay cover in kPa (bottom) during the field test (from 7.00 until 19.00 h).

The lower panel of Figure 5 presents the measured pressure heads in the boulder clay lower cover at four positions in the inner slope. The pressures increase from around zero to positive values, showing a gradual build-up of water pressure in the boulder clay. Two of the curves (of gauges in the lower part of the upper slope) become strongly positive, the other two (of the gauges positioned near the shoulder of the structure) show a more moderate increase. The deformation of the cover was of the order of a few millimetres (not shown). Specialistic investigation at the end of the field test did not show any indication for the presence of a local disturbance in the material or structure of the slope cover near the position of the tensiometers. It was concluded, therefore, that the strong increase in water pressure in the lower part of the lower slope cover is the result of the saturation of the water retaining structure to a level above the intermediate slope cover.

CALIBRATION OF THE PLAXFLOW AND PLAXIS DEFORMATION MODEL TO THE FIELD TEST RESULTS

The geometry of the soil structure as defined by the PlaxFlow module (see Figure 6) was used to calibrate the material parameters to the results of the field test. For the flow calculations, 7.353 linear elements were employed. Variable time stepping was used, with time steps between 5×10^{-4} and 0.5 days with a maximum of ten iterations per timestep. The tolerance of the iteration error was set to the value of 0.01. Overtopping was simulated by imposing a zero pressure on the inner slope. It is noted that the best estimates for the saturated conductivity were close to the measured values (compare the values for saturated conductivity in Tables 1 and 2).

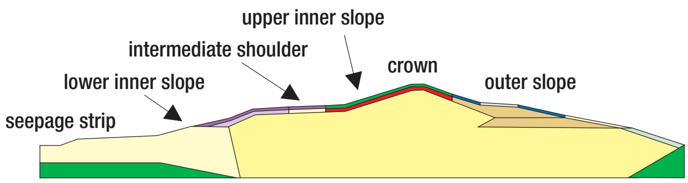


Figure 6: The geometry of the IJsselmeerdijk, as defined for the PlaxFlow and Plaxis calculations.

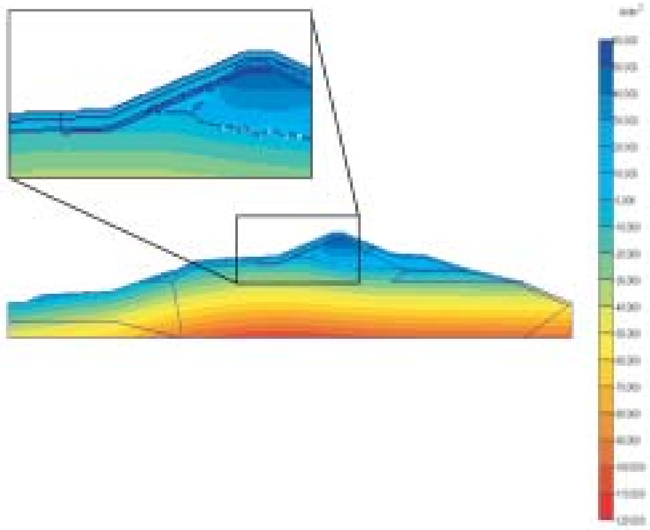


Figure 7: Calculated pressure distribution at the end of the field test.

Figure 7 shows the calculated pressure distribution at the end of the field test. The core of the structure becomes saturated to a maximum height of approximately 1 meter above the intermediate shoulder. The duration to the saturation of the intermediate shoulder in the calculation is in close agreement with the time period after which the air bubbles stopped to appear at the intermediate shoulder in the field test.

Due to hardware problems, the time dependent behaviour of the calculated pressure heads can not be presented here. Instead, the measured and calculated water pressures at the end of the test are shown in Table 4. The calculated values in the upper part of the slope agree reasonably well with the measured values. In the lower part of the slope, the calculated water pressures remain lower than the measured values. Finally, the tensiometers positioned in the sand in the upper part of the core showed an increase in the water pressure. In the calculation this is not the case. The increase of measured water pressure may be related to the presence of a local disturbance with a deviating (lower) permeability. This hypothesis was not further investigated.

Location	Depth [m from surface]	Measured pressure [kPa]	Calculated pressure [kPa]
Upper cover (high)	-0.40	2.2	3.4
Lower cover (high)	-0.75	3.5	5.1
Upper cover (low)	-0.40	5.2	3.1
Lower cover (low)	-0.75	7.2	2.7
Sand core (high)	-1.30	5.6	< 0

Table 4: Calculated pressure in kPa at several locations at the end of the field test.

The deformation measurements were used to calibrate the stiffness parameters of the soil (not tested in the laboratory) in Plaxis V8. For this, the various water pressures as derived with the PlaxFlow model were used as input values. The strength parameters could not be calibrated with the field test, because failure did not take place in this field test. To describe the strength properties of the soil, a Mohr-Coulomb model was assumed.

With Plaxis V8 the safety factor was derived with the c/ϕ-reduction, using 2451 six-node elements on which the water pressure from PlaxFlow was interpolated. For the calculation of the safety factor, a maximum number of 200 steps, and a tolerance of 0.03 were used. The value of the safety factor at complete failure (i.e. deformation exceeds 1.0 m) was employed as the measure here. During the field test, the calculated safety factor decreased from 2.10 at the start to 1.91 after the twelve-hour period of overtopping.

EXTRAPOLATION TO CRITICAL WATER LEVEL AND TO COMPLETE SECTION

The field test was carried out under calm weather conditions, meaning that the lake water level was low. During a critical storm for which the soil structure should be evaluated, the water level will increase (and decrease again) with two meters during a period of 35 hours. Therefore, the field test results were to be extrapolated for this situation. Imposing the water level condition on the base of the lake and the outer slope of the structure did this. In addition, during a period of 12 hours (falling simultaneous with the period of maximum water level), overtopping was assumed by imposing the zero pressure condition at the slope surface again. The water pressures were calculated with PlaxFlow and, subsequently, imported in Plaxis V8.

To extrapolate for the total section, the thickness and saturated conductivity values of the two covers were varied. Table 5 shows the evolution of the safety factor during the critical storm of some of the considered variations. For the best estimate of the hydraulic conductivities of the soil structure, the safety factor equals 1.77. The required safety factor for the section was 1.16.

Variation	Initial safety	Minimum safety
Best guess parameters	2,23	1,77
Thickness lower cover 0.4 m	2,14	2,08
Thickness lower cover 1.2 m	2,31	1,76
Conductivity of upper cover 10 times higher	2,22	1,76
Conductivity of lower cover 2.5 times higher	2,23	1,26

Table 5: Variations in geometry and material parameters for the situation during a critical storm with overtopping, with initial and minimal safety

The minimum safety factor is 1.26, for the situation in which the hydraulic conductivity of the boulder clay in the inner cover is a factor of 2.5 larger than the calibrated value. In this situation, the soil structure became completely saturated.
Increasing the thickness of the lower cover has a positive influence on the stability. However, a thicker than average lower cover does not lead to a significant increase of the safety factor. Increasing the conductivity of the upper cover barely influences the safety factor.

CONCLUSIONS

The assessment led to a positive result for the safety of (the risk of) sliding of the inner slope cover during wave overtopping for the complete section of 8.5 kilometres. In this case, the strength properties of the boulder clay appeared to be crucial to achieve this result. These properties were measured in the laboratory using a simple shear apparatus with drained, consolidated samples under low stress conditions. PlaxFlow proved to be a useful tool to calculate the overtopping situation imposed in the field test and to extrapolate the results to the extreme, normative situation of a 1/2000 years' storm. The combination with Plaxis V8 appeared useful to calculate the safety factor of the structure during the storm.

RECOMMENDATIONS FOR APPLICABILITY OF PLAXFLOW

On the basis of this case study, a few recommendations can be made for the applicability of the PlaxFlow module to study the stabilitiy of water retaining structures:

- Both saturated and unsaturated behaviour of the cover appeared to be very important in this case study. It is recommended to increase the insight in and, at a later stage, to classify the unsaturated properties of the covers of water retaining structures in a systematic way.
- PlaxFlow was shown to be a useful tool to, first, calibrate the water pressures to a field test and, next, to extrapolate the results to critical storm conditions. However, in this case the structure did not show any failure during the field test. Calibration of the combined flow and deformation modules to the results of another field test, in which the slope cover does fail, will reduce the uncertainty in the critical strength of the slope against sliding.
- Constant average flow rate was considered to be a conservative assumption here, for the real situation of a time dependent overtopping flow rate. In combination with a field test, in which a time dependent flow rate is imposed, PlaxFlow can be employed to study (a) the validity and (b) the decrease in the stability factor, as a result of using this assumption. Small flow rates will be sufficient to study the influence on infiltration, bigger flow rates are necessary to combine this study with the slope resistance against erosion.

REFERENCES

[1] GeoDelft, Advanced Assessment of 'Sliding of Inner slope during overtopping' IJsselmeerdijk km 17.5 - km 26.0 (in Dutch), CO401850/68, C. A. Schoofs, and J. M. van Esch, GeoDelft, Delft, Netherlands, 2002.

[2] Delft Cluster, Groundwater Flow Module, Report 10201/70, J. M. van Esch, Delft, 2001.

[3] Van Genuchten, M.Th., A closed-form equation for predicting the hydraulic function of unsaturated soils, Soil Sci. Soc. Am. J., 44, 892-898, 1980.

[4] Schoofs, S., J.M. van Esch, and J.J.H. van den Akker, Measuring and modeling results of unsaturated flow through dikes, Environmental Geomechanics, L. Vulliet, L. Laloui and B. Schrefler (eds), Monte Verità, Switzerland, 283-288, 2002.

COMPARISON OF COMPUTED VS. MEASURED LOAD/SETTLEMENT RESPONSE OF A FOOTING BEARING ON STIFF TO VERY STIFF CLAY

Authors: K. E. Tand, Principal Engineer, Kenneth E. Tand and Associates / Dr. M. W. O'Neill, Ph.D., Professor, University of Houston

INTRODUCTION

A 2.4 meter (7.9 foot) diameter underreamed footing (belled shaft) bearing at a depth of 2.5 meters (8.2 feet) was load tested to failure in axial compression at the National Geotechnical Experimentation Site at the University of Houston in Texas, USA in 1983. The subsoils are stiff to very stiff clay. This article discusses use of the hardening soil model in PLAXIS 8.1 to compute the load/settlement response of the footing.

The results of this load test have been an interest of Dr. Michael W. O'Neill since the test was performed in 1983 due to its unusual behavior. It is very unfortunate for the geotechnical engineering profession that we lost Dr. O'Neill on August 2 of this year. Mike was an internationally known expert in deep foundations, a distinguished professor at the University of Houston for 29 years, and I personally knew him as a man of great integrity. He received many professional awards, and was given the distinguished honor of presenting ASCE's Karl Terzaghi lecture in 1998. I had the personal privilege of working with Mike on this subject, and his expertise and insight are reflected in this article.

SUBSOIL GEOLOGY

The site is located on a Pleistocene age deltaic deposit known locally as the Beaumont formation [1]. The Beaumont is about 8 meters deep at the test site, and it is underlain by an older geologic formation known as the Montgomery formation. The subsoils in both formations are primarily clay with occasional interbedded seams and layers of sand and silt. The consistency of the clays is generally stiff to very stiff, and they have been overconsolidated due to desiccation. The first major silt or sand stratum occurs at a depth of 14 meters (47 feet) which is below the depth of influence of the footing load test.

The water level was at a depth of about 2.3 meters (7.5 feet) below grade at the time of the load test.

After deposition, vertical fissures were formed by shrinkage, and cracks at the surface are sometimes more than 5 cm (2 inches) wide. Soil from the surface was washed down into the cracks during periods of heavy rainfall. The soft sediments in the cracks were then compressed when the clays swelled leaving locked-in horizontal stress. This process was repeated throughout Pleistocene to Modern geologic times, and K_0 values of 3.0 and greater have been measured at this site. The process of desiccation and subsequent rewetting caused cyclic shearing displacements in the clay mass that produced polished failure planes referred to as slickensides. The slickensides are widely variable in size and orientation. The clays are spatially inhomogeneous, and exhibit some anisotropic properties due to their stress history.

The joints and horizontal locked-in stress affect the strength, deformation, and permeability properties of the clays. The soil parameters have been extensively studied at this site and they are summarized in Table 1. A more detailed summary of the database can be found on the web site at www.unh.edu/nges. The laboratory and in situ tests in the database indicate a wide range in the strength/deformation properties of the clays due to the effects of secondary structure, loading stress path and effects of sample disturbance.

FOOTING AND INSTRUMENTATION

The footing load test arrangement [2] is shown on figure 1. The test footing was a 2.4 meter (7.9 feet) diameter underreamed footing (belled shaft) with a 0.76 meter (2.5 feet) shaft. The bearing depth was 2.5 meters (8.2 feet) below grade. In order to permit access to the bell to place instruments, the diameter of the shaft that was initially drilled was 0.91 meters (3 feet). The footing was instrumented with 9 earth pressure cells, 2 strain gauges, and 2 telltales.

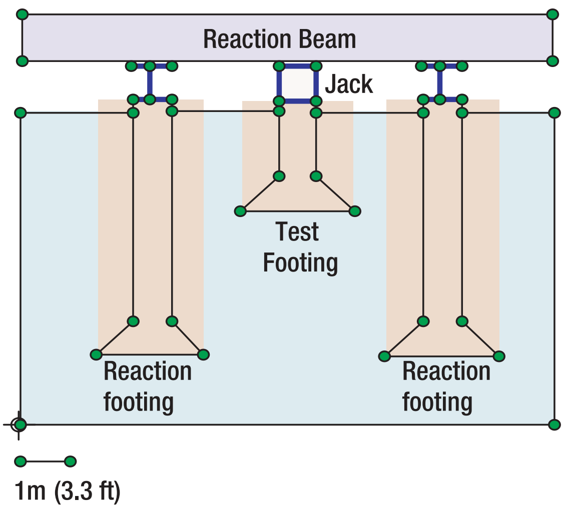


Figure 1: Load Test Set Up

A 0.76 meter (2.5 foot) diameter cardboard casing (sonatube) was set inside the shaft to act as a form to prevent shear from being transferred between the concrete shaft and clay, and a reinforcing cage consisting of eight No. 7 bars with a No. 3 smooth spiral was set inside the casing. When placing concrete in the bell, the concrete rose up 0.23 meters (.75 feet) into the annulus between the cardboard casing and the soil forming a small collar at the top of the underream. The average compressive strength of the concrete was 27.4MN/m2 (3,970 psi) when the footing was load tested.
The reaction system consisted of two underreamed footings placed 2.9 meters (9.5 feet) on each side of the test footing (center-to-center spacing). Their diameter was the same as that of the test footing, but the bearing depth was increased to 5.5 meters (18 feet) below grade.

FOOTING LOAD TEST

The load was applied to the top of the shaft in 445 kN (100 kips) increments and held for sixty minutes to a top load of 2224 kN (500 kips) with a hydraulic jack.

Thereafter, the load was applied in 222 kN (50 kips) increments and held for thirty minutes so that the failure load could be defined clearly.

The footing had to be unloaded at the end of the 3115 kN (700 kips) load cycle to insert shims due to the large displacement. Plunging failure occurred at a load of 3336 kN (750 kips) during the reload cycle. After completion of the load test, the footing was excavated and no indications of structural failure were found.

Settlement was measured using dial gauges attached to an independent reference beam. After initial settlement occurred, time related settlement continued to occur due to a combination of the following factors:

- Creep that occurs without volume changes.
- Volume changes that occur due to consolidation under partially drained conditions.
- Dissipation of suction pressures that developed on top of the underream after applying each load increment due to the propensity for a gap to develop and enlarge above the underream. It is surmised that as suction dissipated, load was being constantly transferred to the base resulting in increasing settlement. The collar around the top of the underream acted as a plug allowing suction to develop.

Initial studies [2], based on results of the earth pressure cells, indicated that about 850 KN (190 kips) of load was being carried in suction (p_s~200 kPa) above the bell and friction on small portions of the bell and shaft in contact with the soil at a load of 1880 kN (400 K). However, footings load tested in uplift at a later date [3] indicate that suction pressures on the base of the footings were substantially less (p_s~70 kPa), and that they dissipated rapidly.

The top of the bell is in a weather zone near the ground surface, and suction developed on the top of the bell should have been less than measured at the base of the footings tested in uplift. Also, suction should have dissipated more rapidly due to the presence of recently active fissures in the weathered zone. It is thus surmised that the earth pressure cells did not register the full bearing pressures until the loading approached failure, perhaps due to arching around the earth pressure cells that had been bedded on sand. The earth pressure cells did register a small increase in bearing pressures at the base of the footings (Δq~19 kPa) during the holding periods indicating that suction did develop and was dissipating, but the magnitude is unknown. However, it is believed that most of the suction dissipated in the 1/2 to 1 hour holding periods before the next load interval had been applied. This subject is currently being studied in more detail.

FE ANALYSIS

The geometric model employed in the FE analysis is shown in fig. 2. The calculations were performed using PLAXIS 8.1 in the axisymmetic mode with 1967 triangle elements with 15 nodes per element. The hardening soil model, as described in the Materials Model Manual, was used to model the stiff to very stiff clays. The hardening soil model is an advanced model for the simulation of soil behavior where the stiffness moduli is stress dependent, and the moduli is reduced with strain according to a hyperbolic relationship. The underreamed footing was modeled using linear elastic properties of the concrete, which is justified considering that there were no cracks found in the excavated footing.

The FE model was configured so that suction forces above the underream were not allowed to develop. The modeling procedure is described as follows:

- The FE model included the footing/soil layers, boundary conditions, etc. that would normally be required to model the load test. A thin layer of soil (dummy layer) was placed directly above the underream. This layer was turned off (deleted) during the calculation phase so that no suction or tension would develop on top of the underream. Also, a thin layer of soil was placed around the shaft in the FE model. This layer was turned off to model the void that was purposely constructed to prevent load from being transferred to the shaft.
- Effective stress parameters (see Table 1) were input for the clay layers. The volume changes in the clays during the loading test were small due to the high degree of overconsolidation, low permeability, and the short duration of the loading test. Thus, the FE computations were performed in the undrained mode.
- The initial stresses were generated assuming a water table at 2.3 meters (7.5 foot) depth, and Ko values from the site-specific geotechnical data.
- The calculations were performed using the stage construction procedure. The measured load was applied as a traction at the top of the footing in one step. In a separate calculation, the load was removed and the rebound computed.
- During the FE calculations, it was found that high effective stresses and pore pressures were developing in a small area next to the collar around the shaft due to stress concentrations. This caused numerical ill conditioning, and the computations collapsed. This problem was solved by increasing the strength in the soil clusters next to the collar, and specifying drained conditions. However, the interface at the collar was manually changed to the original soil cluster so that excessive friction would not be transferred to the shaft.
- The curve program in Plaxis was used to graph the FE computed load/ settlement curve of the footing loading test.

Using the above procedure, there was no attempt to model the increasing settlement that occurred during each holding period due to dissipation of suction. Thus, the FE generated curve is compared with the settlement at the end of each load increment because it was deduced that most of the suction had dissipated at this time.

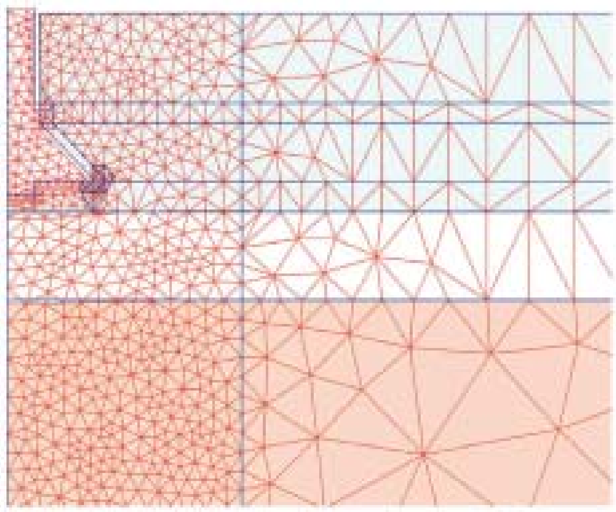


Figure 2: FEM Model

For the initial calculations, soil parameters c', φ' (CIU Bar tests), E' and K_0 were obtained from the referenced database (Table 1). The default values in the soil model were accepted as providing realistic parameters, with the exceptions that the interface factor R_{int} was changed to 0.55 where concrete was in contact with the soil and the power m for the stress dependent stiffness was changed to 0.8. Extensive parameter studies were then performed until good agreement was obtained with the field load/settlement response of the footing. The FE computed load/settlement curves are compared with the field loading test in fig. 3.

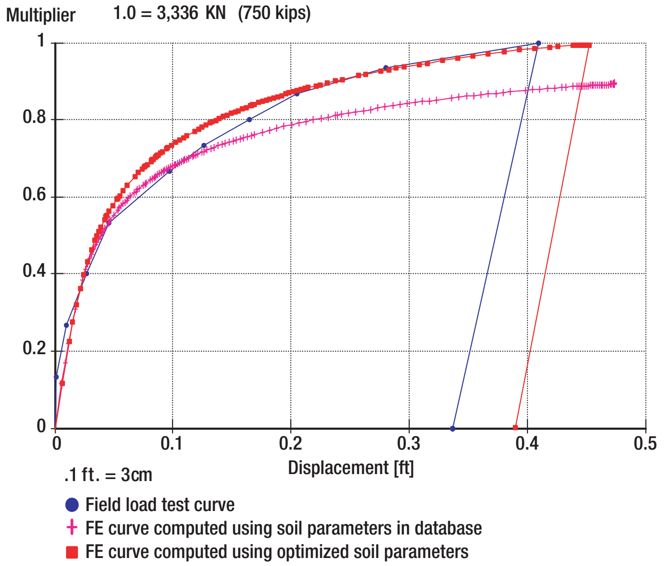


Figure 3: Load/Settlement Curve

Table 1 provides a summary comparison of the back-calculated, or optimized, values of c', φ', E' and K_0 with values reported in the generalized site database.

The following comments pertain to the items marked with asterisks in Table 1.

* E_{50} is not E_{50}^{ref}. It is the value of E_{50} (secant modulus at 50% strain) for the initial conditions computed using eq. 5.2 in The Material Models Manual.

** L-2 was a layer of sandy clay with sand seams that contained numerous calcareous nodules. The cone tip bearing stress qc in this layer was variable but was generally in the order of 4.8 MPa (100 ksf). This site detail and was not addressed in the generalized on-line site database. The sandy clays in this layer are very stiff to hard, and are probably cemented with calcium carbonate. φ' was chosen as 30° which is the peak effective stress angle of internal friction of the sandy clay (L-4), and c' was estimated as 2 times c' of L-4 based on the ratio of qc between these two layers. The site data base also did not consider the increased stiffness of this layer.

The back-calculated (optimized) soil parameters in Table 1 are those values that produced a good match with the measured load/settlement curve, and are within reasonable, but not exact, agreement with the parameters in the site data base. The optimized values of Ko and E' that influence the initial stiffness, which were determined from in situ testing, are in good agreement with those in the site data base. However, c' and φ' that have greater influence at failure are somewhat different. The parameters c' and φ' in the site data base were determined from triaxial compression tests on samples whose strength was affected by secondary structure. In the soil mass, the slickensides and fissures are confined by surrounding soil increasing the strength, and the failure planes during loading do not entirely coincide with the weakened planes. The optimized values determined by FE analysis would thus be expected to represent c' and φ' of the soil mass.

Parameter studies indicated that c' and φ' in layers L-1b and L-3 [both stiff clay] have a significant effect on the FE computed load/settlement curve, and thus provided somewhat of a baseline to compare values of c' and φ' in the site data base with the optimized values. Both CIU bar and CKoU bar triaxial compression tests had been performed on samples from layer L-3, and the optimized values of c' and ∅' were about the average of the two different laboratory testing methods. This may be a coincidence, or it may be that the in situ stress path during the loading test resulted in c' and ' being an average of the laboratory tests with different stress paths.

Considering the complexity of the soil profile and limitations of the constitutive model, the authors believe that the soil parameters are optimal for the site when using the hardening soil model. The optimization process illustrates how geotechnical engineering judgment was required to predict field response from laboratory or in situ tests, not simply accepting results of these tests face value.

The FEM is only a numerical model, or simulation tool, and the hardening soil model does not consider the anisotropic or strain softening behavior of the clay. Good curve matching can also be obtained by making minor changes in soil parameters, i.e. increasing E_s and decreasing K_0. Thus, the optimized soil parameters can only be considered as first order approximations.

Finally, simple FE analyses were performed to evaluate whether uplift on the reaction footings affected the load/settlement response of the test footing. The Plaxis 3D Tunnel program was used for this purpose. The FE model consisted of 30,888 wedge shaped elements, and included adhesion of the clay subsoils to the reaction shafts. The underreamed (round) footings were modeled as equivalent square footings with a wedge face in the x-y plane (axis of all three footings) so that the influence of the upward pressure from the bell shaped reaction footings could be assessed. However, a wedge shape is not allowed in the z plane, and thus this model probably overestimated the effects of the reaction footings.

FE analysis indicates that the computed load/settlement response of the test footing, when analyzed by itself, is somewhat softer than when the reaction footings are included in the FEM model. The difference is about 5 percent when the load is one-half the failure

		U of H Database							**FEM Back-Calculated**			
Soil Layer	Depth meters (feet)	qc MPa (ksf)	Ko	E MPa (ksf)	CIU Bar C' -kPa-	φ' -deg.-	CKoU Bar C' -kPa-	φ' -deg.-	Ko	E50* -MPa- (ksf)	C' -kPa- (ksf)	φ' -deg.-
L-1a	0 - 2.1	0.7	~3.0	11.2	9.5	25	-	-	3.0	11.5	14.4	20
	(0-7)	(15)		(230)	(.2)					(240)	(.3)	
L-1b	2.1-2.7	1.4	~2.5	12.9	9,5	25	-	-	2.5	15.8	28.7	20
	(7-9)	(30)		(270)	(.2)					(330)	(.6)	
L-2	2.7-4.3	4.8	~2.0	14.4	**	**	-	-	1.8	19.2	71.8	30
	(9-14)	(100)		(300)						(400)	(1.5)	
L-3	4.3-7.9	1.9	~1.5	19.1	0	25	57	16	1.5	18.7	33.5	20
	(14-26)	(40)		(400)			(1.2)			(390)	(.7)	
L-4	7.9-15.2	2.4	~1.2	34.0	37	30	0	33	1.2	33.0	37	30
	(26-50)	(50)		(710)	(.8)					(690)	(.8)	

Table 1: Summary of Soil Parameters

load where elastic behavior predominates. However, the difference increases to about 10 percent at three quarters of the failure load. The FE computed failure load is 7 percent greater than the measured load when using the optimized soil parameters. This subject is presently being studied in more detail.

SUMMARY

Several conclusions can be drawn from this study:

- The hardening soil model in Plaxis 8.1 was able to match the field load/ settlement response of an underreamed footing bearing on stiff to very stiff clay within reasonable accuracy. However, parameter studies were required to back-calculate (optimize) the soil parameters input into the FE model.
- The optimized soil parameters for the hardening soil model were similar to those that are posted in the database for the test site, but not identical to those parameters. Engineering judgment guided the selection of optimized final values.
- FE analysis indicates that the earlier interpretation that initial loads had been carried largely by suction on the roof of the bell was probably not valid, and that the earth pressure cells did not register the full bearing pressures until the loading approached failure.
- The 3 D analysis indicates that the reaction footings only had a small influence on the load/settlement response of the test footing. The FE computed load/settlement curve of the test footing with the reaction footings included was somewhat stiffer than the field curve. However, it was in reasonable agreement considering that the footings had to be modeled as equivalent square footings with a wedge face in the X-Y plane, not round bell shaped footings.
- Evaluation and reporting of full-scale field load tests such as the one addressed here are of paramount importance in helping the geotechnical engineering profession understand how to interpret and use routine soil test data.

REFERENCES

1 Mahar, L. J., and M. W. O'Neill, Geotechnical Characterization of Desiccated Clay, Journal of Geotechnical Engineering, ASCE, January, 1983, pp. 56-71.

2 O'Neill, M. W., and S. A. Sheikh, Geotechnical Behavior of Underreams in Pleistocene Clay, Drilled Piers and Caissons, C. N. Baker, Jr., Editor, ASCE, May, 1985, pp. 57-75.

3 Yazdanbod, A., S. A. Sheikh, and M. W. O'Neill, Uplift of Shallow Underreams in Jointed Clay, Foundations for Transmission Line Towers, ASCE Geotechnical Special Publication No. 12, April, 1987, pp. 110-127.

PLAXIS V8: PERMEABILITY OF INTERFACES

In the Plaxis V8 Scientific Manual it is said that an interface is impermeable in groundwater mode if it is switched off, and is permeable if it switched on. In the Reference Manual the exact opposite is said. The Reference Manual is correct: An interface is impermeable if it is switched on in groundwater mode, and is permeable if it is switched off.

PLAXFLOW: AXISYMMETRY

Q: Is it right that PlaxFlow does not allow for axisymmetric calculations?

A: Indeed, in the first release of PlaxFlow, axisymmetry was not included. This omission has been improved in update pack 1 of PlaxFlow, which is available from the Plaxis web site (www.plaxis.nl > user service > downloads). Hence, the most recent version does allow for axisymmetric calculations.

PLAXFLOW: REPRESENTATION OF PORE PRESSURES

Q: Why are tensile pore stresses from groundwater flow calculations in Plaxis V8 cut-off at zero and why not in PlaxFlow?

A: PlaxFlow is meant to model the process of groundwater flow. Therefore, models are incorporated that describe the permeability and the degree of saturation of the soil in relation to the pore pressure. The pore pressure distribution by itself is a result of the groundwater flow calculation and may be influenced by modelling errors. Tensile pore stresses can be realistic to a certain extent, but they are also influenced by modelling errors. In PlaxFlow the 'pure' computational results are shown in the output program (including possible tensile pore stresses). It is the user's responsibility to properly interpret whether the results are realistic or not.

In Plaxis V8, groundwater flow is used as a tool to produce pore pressure distributions for deformation and stability analysis. Tensile pore stresses increase the stability of geotechnical structures. If too large tensile pore stresses from a groundwater flow calculation are used in a stability analysis, the geotechnical structure may be less safe in reality than the calculation indicates. To avoid such a situation, Plaxis V8 cuts-off tensile pore stresses from groundwater flow calculations.

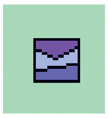
THE INTERACTION BETWEEN PLAXIS V8 AND PLAXFLOW: A SHORT INTRODUCTION

Authors: Wout Broere & Dennis Waterman

UNDRAINED RIVER EMBANKMENT WITH TRANSIENT FLOW (EXTENSION TO LESSON 3 OF THE PLAXIS V8 MANUAL)

The river embankment described in Lesson 3 of the Plaxis V8 manual was loaded by raising the water level, simulated by instantly raising the phreatic levels for the different soil layers. The combination of Plaxis 8.2 and PlaxFlow makes it possible to simulate this problem more realistically by raising the water table against the embankment over time and calculating the resulting time dependent pore pressures in the lower soil layers.

This exercise is based on the same geometry and soil parameters as described in Lesson 3 of the Plaxis Tutorial Manual. To save time, we will not input all data again, but copy the existing geometry and data sets. Open the problem files from Lesson 3 in Input. Select Save As from the File menu, to save the problem under a different name.



To change the soil parameters and boundary conditions for transient flow, click the Initial Conditions button. The toolbar shows a new button, which can be used to open the groundwater flow Material Sets dialog. This button is similar to the standard Material Sets button, but has blue colours.

In this dialog the three data sets are already present. Follow the next steps to specify the additional flow parameters, changing the flow model from the deafult value of Linear to Van Genuchten.

- Click on the Material Sets button and select to edit he first data set.
- On the General tabsheet, select Expert parameter sets
- Click on the (Un)saturated parameters tab
- In the Model combo box, select the Van Genuchten model.
- Check that csat equals 10-4 (1e-4) and yunsat equals 104 (1e4).
- Repeat this for the two remaining data sets.

Generate pore pressures using a groundwater flow calculation Save the problem and go to the Calculations program. Select Phase 2. To clear the previous Staged construction and Pore Pressure settings, right-click the mouse on the phase name and select Reset Staged Construction. Right-click again and select Reset Water Conditions.

To define the transient flow boundary conditions, follow these steps:

- On the Parameters tab, first select Reset Displacements to Zero.
- In the Loading Input box, select Staged construction and enter a Time Interval of 10 days.
- Click the GW Flow button to define the transient flow conditions.
- Select the Water Level tool and draw a water level. This will be a line, consisting of three sections, with coordinates (-1, 10), (20, 10), (45, 10) and (66, 10).
- Select Closed Flow Boundary from the toolbar and close the clay and peat layer on the left side of the mesh and the peat layer on the right side. Draw closed flow boundaries from (0,10) to (0,4) and from (65,7) to (65,4).
- Now select the Selection arrow from the upper toolbar and double-click the left section of the water level (between (0,10) and (20,10)).

A Time dependent head dialog will appear. Select the table option and enter the following values:

Time (day)	Head (m)
0	10
2	13
5	15
10	15

Close the dialog and select Calculate from the toolbar. After the groundwater flow calculation ends, click the Exit button to return to the PlaxFlow program and subsequently the Update button to return to the Plaxis Calculations program. Press the Calculate button to start the deformation calculations.

In the Output program the differences in the results compared with the original exercise from the Plaxis V8 manual can be visualized. One of the new options in Plaxis 8.2 is to plot the incremental displacements, not for the last calculation step but for the entire calculation phase. From the Displacements menu select Phase Displacements, Total Displacements. The results are given in Figure 2. The groundwater head at the end of the calculation has not yet reached a steady state, as can be seen in Figure 3.

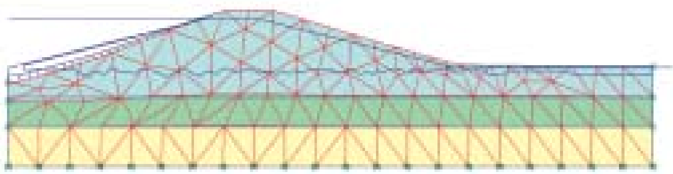


Figure 1: Deformed mesh after 10 days

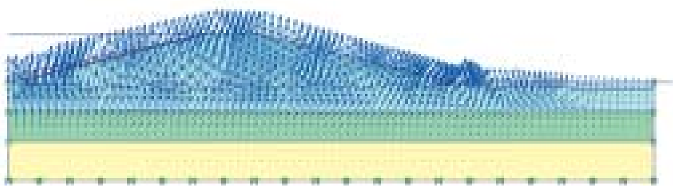


Figure 2: Phase displacements in the last calculation phase

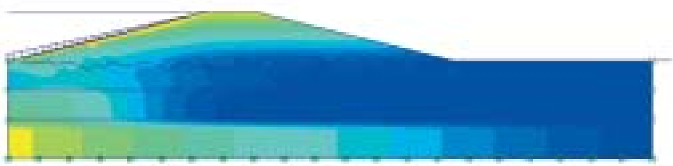


Figure 3: Groundwater head after 10 days

DRY EXCAVATION USING A TIE BACK WALL (EXTENSION TO LESSON 4 OF THE PLAXIS V8 MANUAL)

Lesson 4 of the Plaxis V8 manual describes the dry excavation of a 10 m deep excavation pit, supported by a tie back anchored diaphragm wall. In the original example the water in the excavation is lowered in two phases using a groundwater calculation. In this exercise the same excavation will be modeled using transient flow, in order to study the differences between both cases.

Open the project files from Lesson 4 and save them under a different name. In the Initial Conditions open the groundwater flow material sets dialog box and set all material sets to Expert – van Genuchten. Make sure that csat equals 10-4 and yunsat equals 104 for all soil layers. Generate the initial pore pressure based on a groundwater flow calculation instead of phreatic levels. You will also be asked to generate the initial stresses again, as the pore pressures have changed. After generating the initial stresses, save the problem and go to the calculation program.

In the staged construction definitions, some changes are needed.

Phase 1:

- No changes.

Phase 2:

- Set the time interval to 5 days.

Phase 3:

- Set the time interval to 1 day.

Phase 4:

- Set the time interval to 5 days.
- Press the GW Flow button to open PlaxFlow.
- In staged construction mode, make sure that the interfaces in the lowest (loam) layer are switched off for groundwater flow (double click in the geometry configuration mode).The interfaces should be coloured grey instead of orange.
- In water conditions mode, draw a water level with coordinates (-1,17) – (1,17) – (30,13) – (50,13) – (79,17) – (81, 17) to simulate the dewatering of the excavation. Perform a groundwater flow calculation. This may take several minutes.

Phase 5:

- Set the time interval to 1 day.

Phase 6:

- Set the time interval to 1 day.
- In the GW Flow (PlaxFlow) mode make again sure that the lowest part of the interfaces is switched off and draw a water level with coordinates (-1,17) – (1,17) – (30,10) – (50,10) – (79,17) – (81, 17) to simulate the final dewatering of the excavation. Perform a groundwater flow calculation.

Phase 7:

- Add a new phase to investigate the time dependent effect of the lowered water level inside the excavation. Set the time interval to 90 days. Use the same boundary conditions as in Phase 6 and perform a groundwater flow calculation.

Except for the phases where a transient groundwater calculation is made, the time steps are used only to enable a plot of deformations against time later on.

The results can be compared to those from the original exercise. It can be seen that the deformation and bending moments of the wall are not influenced significantly by the time-dependency of the groundwater flow. In the last stage the deformation of the wall only increases slowly in time.

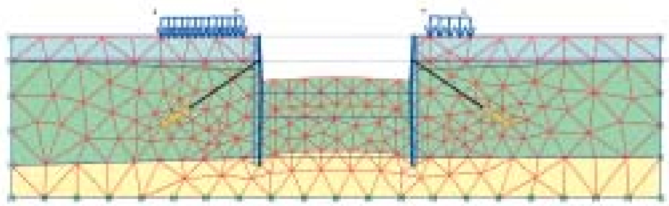


Figure 4: Deformed mesh after Phase 4.

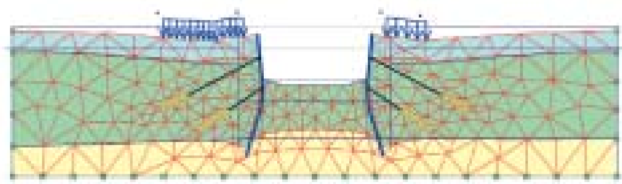


Figure 5: Deformed mesh after Phase 6.

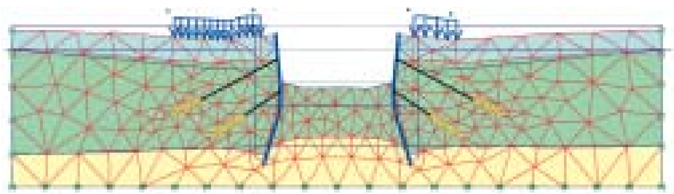


Figure 6: Deformed mesh after Phase 7.

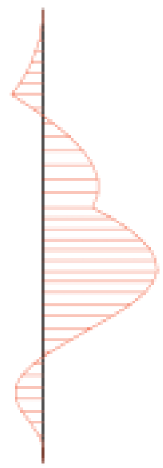


Figure 8: Bending moments in the left diaphragm wall in the final stage

Activities

2003

29 - 31 August, 2003
Short course on Computational Geotechnics (English)
Prague, Czech Republic

17 - 19 September 2003
International Workshop on Geotechnics of Soft Soils
Noordwijkerhout, The Netherlands
For more information see:
http://www.uni-stuttgart.de/igs/SCMEP

28 October 2003
Geotechniekdag
Breda, The Netherlands

27 - 30 October 2003
Curso Internacional de Geomecánica Computacional
Santiago de Querétaro, Mexico

13 & 14 November 2003
European Plaxis users meeting
Karlsruhe, Germany

19 - 21 November 2003
Pratique des éléments finis en Géotechnique
Paris, France

2004

5 - 9 January 2004
Course on Computational Geotechnics & Dynamics
Boulder, Colorado, U.S.A.

18 - 21 January 2004
International course Computational Geotechnics
(English) Noordwijkerhout, The Netherlands

3 - 8 February 2004
GeoSupport 2004
Orlando, Florida, USA

8 - 10 March 2004
Short course on Computational Geotechnics
(German) Stuttgart, Germany

21 - 25 March 2004
International course for Experienced Plaxis Users
(English) Noordwijkerhout, The Netherlands

May 2004
ITA 2004
Singapore

May / June 2004
Experienced Users Course
Boston, U.S.A.

22 - 26 November 2004
15th South-East Asian Geotechnical Conference
Bangkok, Thailand

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