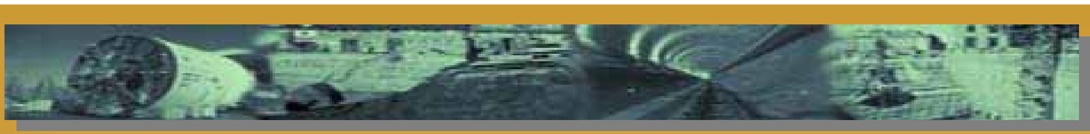

KiKo-Newsletter 2003-2

KiKo

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Zum Geleit:

Zusammenarbeit wird in Zukunft auch durch die EU-Osterweiterung immer wichtiger. Aber vor allem das gemeinsame Arbeiten weltweit hilft uns mit vielen Problemen zurecht zukommen. Das Kiko-Netzwerk besteht nun bereits einige Jahre und gewinnt immer stetig neue Mitglieder. Über 130 Personen zählt unser Kommunikationsnetzwerk mittlerweile. Es vereinfacht eine weltweite Kontaktaufnahme bei Fragen rund um das Bauingenieurwesen. Ehemalige Studierende des Masterstudiengangs “Geotechnik und Infrastruktur” der Universität Hannover werden jedes Jahr zu Vorträgen wiedereingelanden und stellen vor, was sie nach ihrem Master für Tätigkeiten ausüben, aber auch, was ihnen das Studium in Deutschland für Möglichkeiten eröffnete. Im Rahmen dieser Besuche werden jedes Jahr interessante Vorträge über Fragen und Problemlösungen im geotechnischen Bereich gehalten.

Wir bedanken uns bei allen KiKo-Mitgliedern für ihr Engagement an diesem Netzwerk und wünschen weiterhin viel Spaß und Erfolg auf unserer Website.

Das KiKo-Team

Neue Struktur der Kiko-Website

Aus organisatorischen Gründen gibt es seit einiger Zeit keine Kiko-E-mail-Adressen mehr. Künftig werden nur noch private E-mail-Adressen als Kontaktvermittlung genutzt.

Wir bitten alle Kiko-Mitglieder darum, ihre Angaben regelmäßig zu überprüfen und gegebenenfalls zu ändern.

Das Kiko-Team

Finite Element Analysis for Old Barrages with Comparison to Empirical Analysis Methods

B. G. S. Mansour & M. Achmus

Abstract

The piping phenomenon is one of the most important items controlling the design of barrages, especially of the foundation apron length of the structure. This phenomenon depends mainly on the type of soil underneath the foundation and the water level head difference on the structure. In the past it was usual to design the foundation length using the theories of either Lane or Bligh. Nowadays it is common to use the finite-element analysis to estimate the exit gradient value, and hence to judge the safety of the structure according to the soil characteristics.

Through this study a mathematical seepage model for the Assiut Barrage was constructed by means of the finite-element method. The situation and boundary conditions for the Assiut Barrage were modelled thoroughly. The soil characteristics used in the calculations are based on the recent geotechnical investigations undertaken by the Ministry of Water Resources and Irrigation of Egypt in the year 2000. Also through this research a traditional calculation for the creep length under the solid apron of Assiut Barrage was done using both Bligh and Lane theories. A comparison between the three methods mentioned above was made for a better understanding of each method and in order to estimate the maximum water head difference which could be applied to the existing Assiut Barrage.

The old barrages in Egypt are more than hundred years old. Due to the necessity of increasing the water head difference on the barrages -in order to suit the new requirements of irrigation and hydropower generation- it is required either to build new structures or rehabilitate the existing old ones. One method of rehabilitation is to add a low permeability soil blanket on the upstream (U/S)

river bed. This method reduces the hydraulic gradients and allows to increase the water head difference on the structure with respect to the piping phenomenon. Through this study a mathematical seepage model for Zifta barrage as other case study on the River Nile in Egypt is designed by means of the finite-element method. The soil characteristics used in the calculations are based on the recent geotechnical investigations undertaken by the Ministry of Water Resources and Irrigation of Egypt. Also through this research a traditional calculation for the creep length under the solid apron of Zifta barrage will be done using both Bligh and Lane theories. A parametric study for the U/S impermeable clay layer and the soil characteristics underneath the foundation will be examined in the model. A comparison between the three methods mentioned above will be made for a better understanding of each method and in order to estimate the maximum water head difference which could be applied to the existing Zifta barrage. Finally, recommendations concerning suitable rehabilitation options are given.

MSc., Researcher, Institute of Soil Mechanics, Foundation Engineering and Waterpower Engineering, University of Hannover, Appelstr. 9A, 30167 Hannover, Germany; Phone +49 511-762-3529; bgs@stud.uni-hannover.de

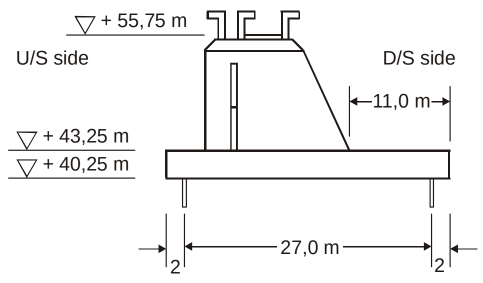
Prof. Dr.-Ing., Head of the Institute of Soil Mechanics, Foundation Engineering and Waterpower Engineering, University of Hannover, Appelstr. 9A, 30167 Hannover, Germany; Phone +49 511-762-4155; achmus@igbe.uni-hannover.de

Description and Current Geotechnical Situation of the Assiut Barrage

The Assiut Barrage is one of the seven working grand barrages on the River Nile in Egypt. It lies between the Naga Hammadi Barrage and the Delta Barrages at km 544.78 downstream (D/S) of Aswan City. The barrage consists of 110 vents with lengths of 5.00 m each. Originally, the barrage was designed to stand a head of 2.50 m during the period of low River Nile water level, and there was no suggestion of holding up the water during the flood period. The maximum head allowed was increased after the completion of the work to 3.00 m during the low water level season. Also, in order to provide additional supplies for the Ibrahimia Canal, a head of 3.20 m was sanctioned during the period of the falling water level.

During 35 years of use of the barrage, and before the remodelling, a certain amount of damage occurred, including deterioration of the masonry floor and piers, cracks in the arches and in the abutments and D/S scour holes, which means the start of the piping phenomena, or due to surface water erosion (mainly due to the short length of the apron foundation against the applied water head). These previously mentioned observations led to quick maintenance work, such as filling the scour holes on the D/S bed of the river with rubble masonry, repairs to the solid floor through the vents using concrete and/or rubble masonry, and in 1928 placing a block work apron 10.00 metres in length on the D/S side to save the foundation. This remedial work saved the barrage and postponed its aging, but it was necessary to increase the water head difference for more storage of water to suit the increasing irrigation demands. For this reason a major project took place in 1934 to rehabilitate the barrages to accommodate a water head difference of 4.2 m. (Figure 1) shows the longitudinal sections of the barrage before and after remodelling.

Old Assiut Barrage (built in 1902)



Current Assiut Barrage (remodelled 1938)

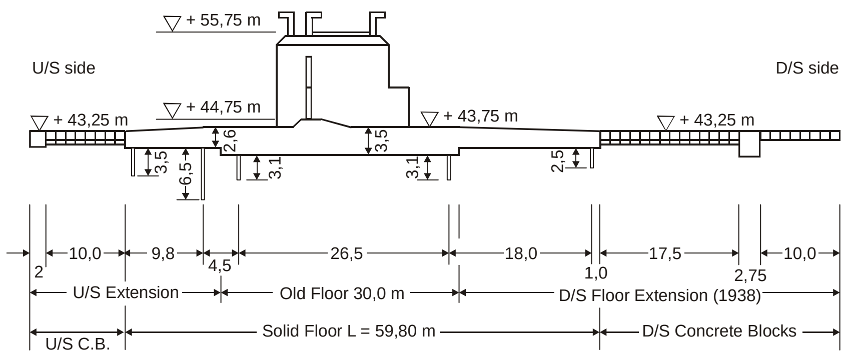


FIGURE (1) Longitudinal Sections of the Assiut Barrage Before and After Remodelling

In the year 2000 a Feasibility Study was started to investigate the rehabilitation work needed to increase the storage capacity U/S of the barrage and to investigate the possibility of adding a hydro-power station to the barrage. Eight different piers were chosen to implement the drilling boreholes. Each borehole was 30 m long (15 m in the barrage body and 15 m in the soil underneath the foundation). Samples were obtained from the piers' material using double core barrels. Disturbed and undisturbed samples from the soil were also taken. From the borehole descriptions the foundation material encountered below the Assiut Barrage structure was shown to consist in the main of fine to coarse sand. The permeability of the soil was measured by falling head permeability tests which were performed through the installed piezometers in the drilled holes; the average permeability was found to be K_{av} = 2.5 x 10^{-5} m/s (Assiut Barrage Joint Venture 2001).

The river bed on the U/S side was shown to consist of a clayey silt layer which has been generated over a long period due to the hydraulics of the river and the sedimentation on the U/S side. The investigations showed that this layer is around 1 m in thickness in some areas. From experience with the River Nile, a deep clay layer should always be found at depths of 20 to 30 m (Leliavsky 1965), but during the Assiut Barrage feasibility study no such deep investigation was undertaken, so the exact location of this layer has not yet been determined.

Description and Current Geotechnical Situation of the Zifta Barrage

Zifta barrage is located on the Damietta branch of the Nile at Km 1046 D/S Aswan city (about 100 Km north of Cairo City). The barrage consists of 50 vents with width of 5.0 m each. Each vent is equipped with double leaf gates operated mechanically by means of two gantry cranes. A navigation lock, 64 m by 12 m chamber, is connected to the barrage at the west side.

Old Zifta barrage was built in the period between 1901 and 1902 in order to raise the River Nile water level to feed the two canals U/S of the barrage. As shown in (Figure 2), the old barrage was built on a plane concrete foundation (32.1 m length and 3 m thickness). The U/S protection consists of 20 m dry pitching above a 13.9 m long clay layer. In the D/S side there was also a dry pitching protection of about 16.5 m length. Two steel sheet piles at the U/S and D/S sides of the solid floor were installed with depths of 2.1 m each. According to the theories and the experience of designers during this period of time, Old Zifta barrage was designed to stand a water head difference of 4 m but then the actual head was 3.77 m (Leliavsky 1955).

Due to some deterioration of the barrage and the D/S bed and also to increase the water head difference, Zifta barrage was remodelled 50 years after it was originally built. In 1952 this remodelling work was finished and as shown in Fig. 1 it included the following main items:

- Extension of the solid floor in the longitudinal direction to be 44.7 m instead of 32.1 m.
- Lengthening the piers and increasing the width of the road above the barrage.
- Increase the solid floor concrete thickness to 3.5 m.
- Install new sluice gates and change all the mechanical equipments of the barrage.
- Add new 10 m D/S protection work consisting of plane concrete blocks resting above the old protection work (dry pitching from crushed lime stones) to be used as D/S filter.
- Install new sheet pile in the D/S side of the new solid floor with 5.5 m depth.

After this remodelling work the water head difference which was applied to the barrage was 4 m which is kept till the time being on this level.

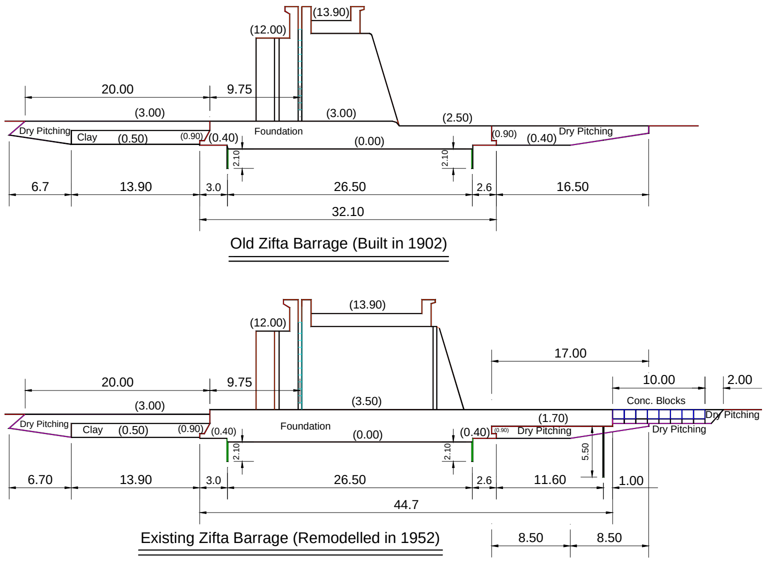


FIGURE (2) Longitudinal Cross Sections of Old and Existing Zifta Barrage

The Ministry of Water Resources and Irrigation in Egypt carried out several investigation programs to the barrage including material and soil investigations. In general, the soil underlying the foundation level in the area of the barrage consists mainly of fine grained sand with sporadic clay lenses. (Table 1) shows a conclusion of the soil stratification at the barrage site according to all the previous investigation programs.

TABLE (1) Soil Stratification & Permeabilities at Zifta Barrage Site

Level (m)		Soil Description	Soil Description
From	To		
+6.00	0.00	Clayey silty fine sand	
0.00	-6.00	Poorly graded medium sand	1×10^{-5}
-6.00	-17.00	Medium to coarse sand	1×10^{-4}

As a conclusion, the soil under the barrage foundation can be assumed as a fine silty sand with an average horizontal permeability of 5.5×10^{-5} m/s. Due to the effect of the clay lenses the vertical permeability can be assumed to be 1.57×10^{-5}. The factor of $K_x = 3.5\ K_y$ was chosen due to previous experience with River Nile barrages (Naga Hammadi Report 1993). The anisotropy factor can be estimated by calibrating the seepage model with the piezometer readings U/S and D/S of the barrage. It was not possible to install piezometers U/S and D/S of

Zifta barrage due to the difficult geometry of the barrage, but during the Naga Hamadi barrage feasibility study this installation was possible, and hence the seepage model was calibrated and an anisotropy factor for the sand layer was found to be 3.5. Due to the very similar soil characteristics of the two locations this factor was also used for the presented calculations for Zifta barrage. No investigations were carried out to the U/S clay layer and to the D/S filter layer. In the following calculations a parametric study for the permeability and the geometry of the clay layer and the D/S filter layer will be done.

Bligh's Method

The length of the apron represents the most important part for the design of the hydraulic structures. The design of the apron depends on the possibilities of percolation in the porous soil upon which the structure is built. Bligh published his theory in 1012 (Bligh 1912). He assumed as an approximation that the hydraulic slope or gradient is constant through the contact length between the structure and the soil. The length is known as the "creep length" and is denoted by the symbol *L*. According to the Bligh theory, the apron will be safe against piping if the ratio is not less than a certain value ($C_{B,crit}$) depending on the type of the soil underneath the apron, see (Table 2). According to practical experience (Leliavsky 1965), the values of C_B in most of the Egyptian soils in the River Nile area and the Delta range between 18 and 24.

TABLE (2) Values of Bligh Coefficient $C_{B,crit}$ for different soils (Bligh 1912).

Type of Soil	$C_{B,crit}$ [-]	Type of Soil	$C_{B,crit}$ [-]
Very fine sand or silt	18	Gravel and sand	9
Fine sand	15	Boulders, gravel, sand	4 to 6
Coarse sand	12		

When applying Bligh's theory to a hydraulic structure equipped with sheet piling, Bligh assumes that the creep length is lengthened by double the length of the sheet pile. This means that during calculations of the creep length, the depth of each cut-off is multiplied by 2. Bligh fixed a limit to this theory for practical application, as it is not possible to consider all sheet pile walls if they are close to each other. He found that this theory can be applied only if the distance between the cut-offs is greater than double the depth of the cut-off.

Lane's Method

In the thirties of the last century, Lane proved that this assumption of a factor 2 "used by Bligh" is not too large, but it is too small, and he suggested using a factor of 3 or 4 to obtain more suitable values for this application (Lane 1935). The seepage formula according to Lane is , where L_v and L_H are the vertical and horizontal lengths and H is the water head difference. The critical ratios $c_{L,crit}$ reported by Lane are given in (Table 3).

TABLE (3) Values of the Lane coefficient $C_{L,crit}$ for different Soils (Lane 1935).

Type of Soil	$C_{L,crit}$ [-]	Type of Soil	$C_{L,crit}$ [-]
Very fine sand	8.5	Coarse gravel w. stones	3
Fine sand	7	Boulders, gravel, stones	2.5
Medium sand	6	Soft clay	3
Coarse sand	5	Medium soft clay	2
Fine gravel	4	Stiff clay	1.8
Medium gravel	3.5	Very stiff clay	1.6

Finite-Element method

The analytical solution is only possible for simple groundwater flow problems, and numerical solutions are required for calculating complicated systems. There are two main methods for carrying out the numerical solutions using the finite methods. The first method is the Finite-Difference Method (FDM) and the other method is the Finite-Element Method (FEM). Here the latter method was applied using the GGU-GW2 programme (Buß 1997).

The differential equation for solving the vertical plane groundwater flow problem in saturated soil is as follows:

$$K_x * \partial^2 h / \partial x^2 + K_y * \partial^2 h / \partial y^2 + Q = 0$$

K_x, K_y Soil permeabilities in *x* and *y* directions.
h Piezometric head.
Q External discharge.
x, y Two-dimension coordinates (*x* horizontal, *y* vertical direction).

The equation is solved numerically by dividing the whole area in which flow appears into a large number of smaller triangles to construct the finite elements required. Within these triangles simple, generally linear, approximation functions are used. The actual, complicated, whole solution is pieced together like a mosaic from the many simple partial solutions. Equation systems result, in which the number of variables corresponds to the number of system nodes. A constant approximation of velocities results from the linear approximation of the piezometric heads inside each element. In order to achieve a better quality of approximation of the velocities, the velocities for the calculation of flow lines are averaged to node values in a follow-up calculation. The solution procedure is that for each element node the velocities in the neighbouring elements are added, and then divided by the number of neighbouring elements. Doing this, the velocity course can be presented in a better way. At the boundary nodes the results are naturally not quite as exact. The approximation of velocities in the region of element nodes with differing material types can be worsened in this way. If the velocities at such boundary nodes are of great interest, it is preferable to refine the element grid in this area. For the calculations presented here a very fine element grid was used to ensure good quality of the solutions. The main output of the FEM analysis is the exit gradient value which is used to judge the safety of the structure against piping by comparing this value with the allowable values as shown in (Table 4) for the different soils given by Khosla (Novak 1992).

TABLE (4) Allowable exit gradients (Novak 1992)

Type of Soil	Allowable Exit Gradient [-]
Fine sand	1 in 6 – 1 in 7 (0.17-0.14)
Coarse sand	1 in 5 – 1 in 6 (0.2-0.17)
Shingle	1 in 4 – 1 in 5 (0.25-0.2)

Assiut Barrage Model Construction and Parametric Study

As the aim was to construct a very detailed finite-element model the situation and boundary conditions for the existing Assiut Barrage were modelled thoroughly. The model includes the U/S (U/S) and D/S (D/S) concrete blocks to study their effect on the exit gradient values and to make a comparison with the different situations if the concrete blocks are neglected. The exit gradients were calculated in two positions for all the models. The first position is just D/S of the solid floor (under the D/S concrete blocks), and the second location is at the D/S side of the D/S concrete blocks. It was found that removing any of the three sets of concrete blocks does not make much difference in the values of the exit gradient, which means that the concrete blocks are more effective for erosion protection. Removing any of these blocks increases the value of the water seepage under the barrage.

The main step of the finite element mesh in the *x*-direction is 10 m and is reduced to 4.5 m and 2.5 m in the area under the barrage and under the concrete blocks. The step in the direction of the *y*-coordinate is 5 m. A parametric study for the size of these elements was carried out to obtain the best size of the elements, and it was found that further refining of the mesh gave almost the same results, so in order to simplify the model the element sizes mentioned above were used in all the next runs of the model. Four types of soil properties were used, as shown in (Table 5).

TABLE (5) Soil properties used for finite-element calculations.

Soil No.	K_x [m/s]	K_y [m/s]	Designation
1	1.00×10^{-50}	1.00×10^{-50}	Solid barrage body
2	1.00×10^{-2}	1.00×10^{-2}	Filter under blocks
3	2.50×10^{-5}	7.14×10^{-6}	Sand
4	2.50×10^{-7}	2.50×10^{-7}	Clay

As shown in the above table, an anisotropy factor of 3.5 was taken for the permeability of the sand layer. This factor was chosen dependent on previous experience with River Nile barrages (Naga Hammadi Report 1993). The anisotropy factor can be estimated by calibrating the seepage model with the piezometer readings U/S and D/S of the barrages. It was not possible to install piezometers U/S and D/S of the Assiut Barrage due to the difficult geometry of the barrage, but during the Naga Hamadi Barrage feasibility study this installation was possible, and hence the seepage model was calibrated and an anisotropy factor for the sand layer was found to be 3.5. Due to the very similar soil characteristics of the two locations this factor was used here. In addition, a parametric study for different possible anisotropy factors was examined (Figure 3). As shown in the figure, it seems very important to calibrate each seepage model with U/S and D/S piezometers to obtain the exact local value of the anisotropy factor. This strongly proves the need for sufficient geotechnical investigations and instrumentation in projects similar to the Assiut Barrage in order to obtain a better judgement of the condition and the behaviour of the structure.

Several models were examined to see the effect of the U/S clay blanket which was generated due to the river’s hydraulics. It was found that without such a layer the exit gradient increases. As it is possible for this layer to be removed through the change of the river’s hydraulics or any dredging of the river bed, so all the next models were generated without this layer. The boundary condition of the model was also examined by varying the depth of the deep clay layer from 23 m to 42 m, and it gave a significant change in the exit gradient values (the deeper this layer is the higher the exit gradient is). As no deep geotechnical investigation was done in the barrage location it was assumed that this deep clay layer is found down to around a depth of 40 m according to previous experience with Naga Hammadi project.

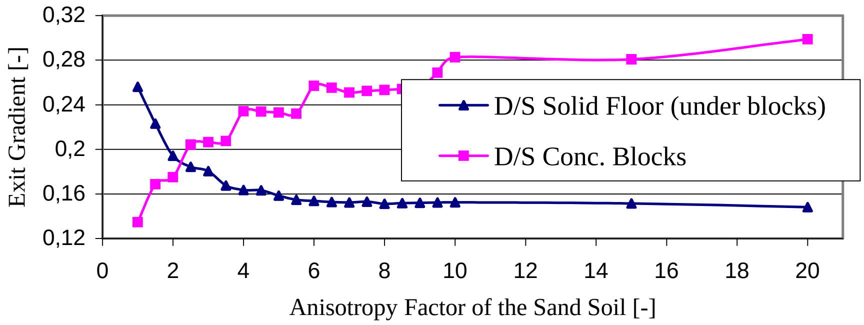


Figure (3)Parametric Study for the Influence of the Sand Anisotropy

The cross section of old Assiut Barrage was used in these calculations (Figure 1). The reason for carrying out these calculations is that the original old barrage contains only two sheet piles, and so an easier comparison of the methods can be obtained. The calculation was done using the Bligh, Lane and Finite-Element methods. The calculation was carried out with regard to only the exit gradients and the creep lengths, and hence to the allowable water head difference on the barrage. It has to be stated that this head difference must be checked with respect to other design factors like uplift. The results of all the calculations are summarized in (Table 6).

The calculation of the creep length using the Bligh and Lane methods was based on the coefficient values for the very fine sand (Tables 2, 3). The reason for choosing these high values, although the recent soil investigation classified the foundation soil as fine to coarse sand, is due to long-term experience with the River Nile. The finite-element model was constructed according to the previously mentioned factors: soil properties and boundary conditions. It should be mentioned that in this model the left boundary is located 112 m U/S of the U/S edge of the solid foundation, and the D/S boundary 130 m D/S of the D/S edge of the solid floor.

By comparing the above results it is clear that the Bligh and Lane methods do not consider the actual behaviour of the seepage under the structure. The two methods deal only with the depth of the sheet piles regardless of their locations. However, the finite-element method shows many differences due to the location and the existence of the sheet piles.

TABLE (6)Comparison of the Maximum Permissible Water Head Differences using the Bligh, Lane and FEM Methods for the Old Assiut Barrage

Method	**Two sheet piles assumed to function**	**U/S sheet pile assumed not to function**	**D/S sheet piles assumed not to function**	**No sheet piles**
Bligh ($C_{B,crit}$ =18)	2.68 m	2.34 m	2.34 m	2 m
Lane ($C_{L,crit}$ =8.5)	3.3 m	2.6 m	2.6	1.9 m
FEM (allowable exit gradient =0.17)	3.3 m	3.35 m	2.6 m	2.8 m
FEM (allowable exit gradient =0.14)	2.7 m	2.8 m	2.15 m	2.3 m

As explained above, in the early years of the old barrage operation severe damage and scour holes occurred, which may be due to the start of the piping phenomena. The barrage was operated with a 3.2 m water head difference during this period. As shown in the results (Table 6), two values for the exit gradients were used as limits. It appears that the higher value (0.17) is critical for the soil of the River Nile, and so it is recommended to use the lower limit (0.14 in the case of fine sand) during design using the finite-element method. The upper limit of 0.17 can be used for carrying out parametric analysis by assuming failure of any sheet piles or increasing the permeability of the soil.

Seepage analysis of the existing Assiut Barrage

The finite-element model was generated thoroughly with all the U/S and D/S concrete blocks and with all the sheet piles as shown in (Fig. 1). Several runs of the model were made to investigate the following cases: all sheet piles (SP) are assumed to function; the two middle old SP not to function; U/S and two old SP not to function; U/S and D/S and two old SP not to function; D/S and two middle old SP not to function. In all the above cases the same procedures for the model parameters as explained above were carried out, and the horizontal permeability was taken as $K_x = 2.5\ 10^{-5}$ m/s = 3.5 K_y. Another run was performed to examine the higher permeability of $K_x = 4\ 10^{-5}$ m/s as obtained from the Hazen formula. In this case it was assumed that all the sheet piles functioned.

When calculating using the Bligh and Lane methods, the three U/S SP were assumed to be as one SP with the greater depth of 6.5 m, due to the fact that they are spaced with a distance of less than double the depth of 6.5 m, as explained above. The exit gradient obtained by the FEM was compared in the normal case with 0.14, and for extreme cases with 0.17, as stated above. The results of the calculations are shown in (Table 7) and (Figure 4) (FEM results). the position of each sheet.

It is clear that the empirical methods do not show the effect caused by the position of each sheet pile and the FEM method takes this effect into consideration. The D/S sheet pile has a greater effect on the value of the exit gradient, which proves the practical experience as stated by designers of such structures (Leliavsky 1965).

The existing barrage has been holding a head difference of 4.2 m since 1936 and has shown no sign of any piping or surface erosion since that time. This value is the same as obtained from our research in the case when all sheet piles are functioning, and compared with the lower limit of the allowable gradient value. However, we suggest that a higher value of head difference can be applied to the barrage for a short time (for example for about ten days, which is needed to store water released from High Aswan Dam in case of sudden rainfalls in the north of Egypt) without risk of damage. In this case we see that it is reasonable to assume the higher value of the gradient limit and to account only for the probability of Old+U/S SP not functioning, then we can apply a 4.65 m head difference to the barrage. This increases the storage capacity of the barrage, and can save a huge amount of water normally released to the sea.

TABLE (7) Comparison between the Maximum Permissible Water Head Difference using the Bligh, Lane & FEM methods for the Existing Assiut Barrage

Method	**All SP funct.**	**Old SP not funct.**	**Old+U/S SP not funct.**	**Old+D/S SP not funct.**	**All SP funct., but K_x =4 10^{-5} m/s = 3.5 K_y**
Bligh ($c_{B,crit}$ =18)	4.9 m	4.55 m	4.28 m	4.28 m	4.9 m
Lane ($c_{L,crit}$ =8.5)	5.68 m	4.96 m	4.37 m	4.37 m	5.68 m
FEM (allowable gradient =0.17)	5.05 m	5 m	4.65 m	3.4 m	4.65 m
FEM (allowable gradient =0.14)	4.2 m	4.1 m	3.7 m	2.8 m	3.7 m

It should be noted that for the calculation of the exit gradients two locations were considered, one under the blocks and D/S of the solid floor and the other D/S of the D/S concrete blocks. The higher values are plotted in the figures given. Normally, the higher value is located D/S of the D/S concrete blocks, but when the D/S sheet pile is removed the higher value appears under the blocks and D/S of the solid floor.

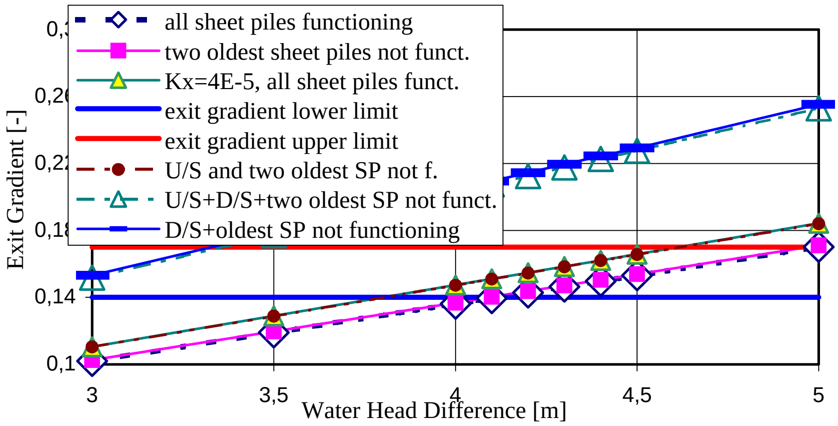


FIGURE (4) Relation between Exit Gradients and Head Difference for the Existing Assiut Barrage (FEM results)

Zifta Barrage Model Construction and Boundary Conditions

The model includes the U/S dry pitching which is located above the 13.9 m length clay layer and the D/S concrete blocks which are located above the old filter with the dry pitching material. The exit gradients were calculated at two locations for all the models. The first location is just D/S of the solid floor (under the filter material which is located under the D/S concrete blocks), and the second location is at the D/S side of the D/S concrete blocks.

The main step of the finite element mesh in the *x*-direction is 10 m and is reduced to 2.5 m in the area under the U/S clay layer and under the D/S filter and concrete blocks. The horizontal step under the main middle part of the barrage foundation is reduced to 5 m. The step in the direction of the *y*-coordinate is 5 m. A parametric study for the size of these elements was carried out to obtain the best size of the elements, and it was found that further refining of the mesh gave almost the same results, so in order to simplify the model the element sizes mentioned above were used in all the next runs of the model. An example for the mesh and one output showing the isolines are shown in (Figure 5). Four types of soil properties were used, as shown in (Table 8). In order to simulate the solid body of the barrage, the concrete blocks and the sheet piles, a very low permeability was thus chosen for soil No. 1 which represents these construction elements. With regard to experience with the Naga Hamadi barrage, an anisotropy factor of 3.5 was taken for the permeability of the sand layer as explained above.

TABLE (8) Soil Properties used for Zifta Barrage Finite-Element Calculations

Soil No.	K_x [m/s]	K_y [m/s]	Designation
1	1.00×10^{-50}	1.00×10^{-50}	Solid barrage body
2	1.00×10^{-2}	1.00×10^{-2}	Dry pitching (Filter)
3	5.50×10^{-5}	1.57×10^{-5}	Sand
4	2.50×10^{-7}	2.50×10^{-7}	Clay

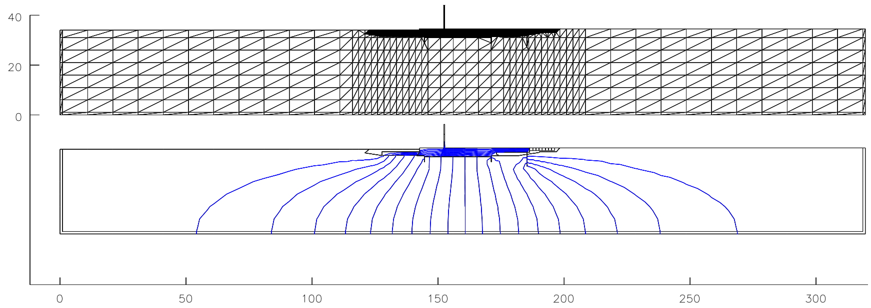


FIGURE (5) Example for the used Mesh and One Output Showing the Isolines

The length of the simulated model is 320 m and the lower deep boundary clay layer is chosen to be on depth 35 m from the River bed. This depth was not verified by a deep borehole but according to previous experience with the River Nile it is known that this layer should be found always in depths 20-30 m below the River Nile bed (Leliavsky 1965). Also this layer was verified during New Naga Hamadi barrage Project and was found in a depth around 40 m from the bed of the River Nile. During previous research by the authors on Assiut barrage, it was found that the location of this deep clay layer causes variations for the exit gradient values at some locations (Achmus and Mansour 2002). The upper boundary of the model is the River bed and the barrage foundation which were used to identify the U/S and the D/S water head to cause the water seepage. A parametric study for the effect of the U/S clay layer as a main effective boundary condition was done and will be shown in the following items. Also the effect of the D/S filter layer was examined by another parametric study as it is believed to be a very important boundary condition affecting the seepage model.

Effect of the D/S Filter Material on the Seepage Analysis Results for Zifta Barrage

The model was established with exactly the cross section of the existing Zifta barrage shown in (Figure 2) and with the boundary conditions and soil properties explained above. In this model the existing boundary condition of the filter layer at the D/S side was considered, the maximum thickness of this filter layer reaching about 1.5 m just D/S the D/S sheet pile (SP). Two locations to measure the exit gradient were chosen, the first just D/S the D/S SP and the second D/S the concrete blocks (CB). It was found that the gradient under the filter layer (just D/S the D/S SP) is very high while the exit gradient D/S the concrete blocks is very small. These previous results showed the important effect of the thickness and the properties of the D/S filter layer and led us to carry out a parametric study for this layer. The results and the description of these parametric studies are shown in (Table 9).

The results given in (Table 9) show that the filter layer attracts the seepage flow D/S the barrage significantly. The higher the thickness of the D/S filter the higher is the gradient below the filter and the lower is the gradient D/S the filter and the concrete blocks. Thus it is evident that two locations to determine the exit gradients for such structures are very important. The first location is below the filter and in this case this gradient value should be below a certain limit to satisfy the hydraulic criteria of the filter. Also of course the filter should satisfy the geometric criteria (Terzaghi 1967& 1922, USBR 1947, USBR 1963). The second location to measure the exit gradient is D/S the CB and in this case it should not exceed the limits given in (Table 4). Unfortunately for this case study there are no data or previous investigations which give the properties of the filter layer and so we can not get a conclusion on the behaviour of the barrage with this thick D/S filter layer.

Also we should mention here that using any empirical method as Lane's or Bligh's does not include the effect of this layer or the effect of the different permeability values of the soil under the foundation and so does not give a real simulation to the seepage under the barrages. It is recommended for any further analysis to Zifta barrage that a detailed investigation of the filter layer should be done and to continue this analysis to get the best rehabilitation option for the barrage.

Model No. 6 mentioned in (Table 9) shows that when reducing the permeability of the filter layer until it reaches the same as the soil under the barrage then the exit gradient D/S the concrete blocks starts to be higher than under the blocks.

TABLE (9) Results of a Parametric Study for the D/S Filter Material

No.	Description of the Model	Gradient Value		Seepage ($m^3/m/s$)
		D/S SP	D/S CB	
1	Existing Zifta barrage cross section, water head = 5.5 m	0.3214	0.0939	7.91014×10^{-5}
	As above but water head (H) = 4.0 m	0.2411	0.0683	5.75283×10^{-5}
	As above but H = 3.0 m	0.1808	0.0512	4.31473×10^{-5}
2	The U/S clay layer is removed, H = 5.5 m	0.3577	0.1013	8.46059×10^{-5}
	As above but H = 4.0 m	0.26	0.0737	6.15316×10^{-5}
	As above but H = 3.0 m	0.195	0.055	4.61487×10^{-5}
3	No U/S clay layer, no U/S part of the filter (which is located U/S the D/S SP), H = 5.5 m	0.3231	0.0915	8.40384×10^{-5}
	As above but H = 4.0 m	0.2349	0.0665	6.11189×10^{-5}
	As above but H = 3.0 m	0.1761	0.0499	4.58391×10^{-5}
4	No U/S clay layer, no U/S part of the filter (which is located U/S the D/S SP), no lower part of filter which is D/S the D/S SP (i.e. filter thickness = 0.5m), H = 5.5 m	0.2008	0.1	8.29881×10^{-5}
	As above but H = 4.0 m	0.146	0.0709	6.0355×10^{-5}
	As above but H = 3.0 m	0.1095	0.0532	4.5266×10^{-5}
5	No U/S clay layer, no all D/S filter layer, no all D/S CB, h = 5.5 m	0.18	-	8.15779×10^{-5}
	As above but H = 4.0 m	0.124	-	5.93294×10^{-5}
	As above but H = 3.0 m	0.0931	-	4.4497×10^{-5}
6	Existing barrage cross section with U/S clay layer and all D/S filter and blocks, $K_{filter} = 1 \times 10^{-1}$ m/s, H = 4.0 m	0.2451	0.0694	5.76529×10^{-5}
	As above but $K_{filter} = 1 \times 10^{-2}$ m/s, H = 4.0 m	0.2411	0.0683	5.75283×10^{-5}
	As above but $K_{filter} = 1 \times 10^{-3}$ m/s, H = 4.0 m	0.1535	0.0603	5.66148×10^{-5}
	As above but $K_{filter} = 1 \times 10^{-4}$ m/s, H = 4.0 m	0.0785	0.105	5.40507×10^{-5}
	As above but $K_{filter} = 5.5 \times 10^{-5}$ m/s, H = 4.0 m	0.0785	0.105	5.32589×10^{-5}
	As above but $K_{filter} = 1 \times 10^{-5}$ m/s, H = 4.0 m	0.0262	0.105	5.11496×10^{-5}

The presented analysis shows that using a thick filter layer D/S the existing barrage can be used as a rehabilitation option to reduce the exit gradient D/S the blocks and increase it under the filter. The filter has to be designed to suit both geometric and hydraulic criteria. With that, the water head difference can be increased on the barrage with regard to the design of piping and erosion. Of course other analyses for the structural elements and the uplift should be done also. Also implementation of a D/S filter with the D/S concrete blocks above the filter will reduce the erosion caused by surface water flow.

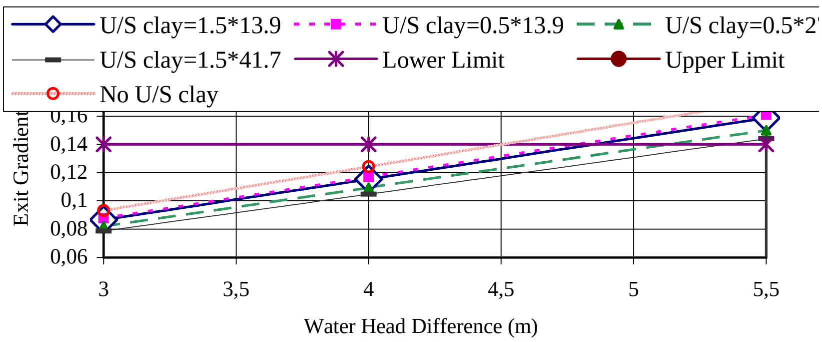
Effect of the U/S Clay Layer on the Seepage Analysis Results for Zifta Barrage

From the results given in (Table 9), models no. 1 and 2, it is evident that the U/S clay layer has a significant effect on the seepage model results. For example, for the case of H = 4 m the gradient value under the filter after removing the clay layer increased from 0.2411 to 0.26 and D/S the CB from 0.0683 to 0.0737. In order to carry out a parametric study concerning the effect of the U/S clay layer as a rehabilitation option for such barrages, some modification of the existing cross section of Zifta barrage were made. The D/S filter layer and the D/S concrete blocks were removed to simplify the model and to have one value only of the exit gradient D/S the solid floor and to avoid the effect of the D/S filter layer as explained in the above section. The results and the description of this parametric study are shown in (Table 10), the dependence of the exit gradient on the water head difference is plotted in (Figure 6).

TABLE (10) Results of a Parametric Study for the U/S Clay-Layer for Zifta Barrage

No.	Description of the Model	Exit Gradient (D/S Solid Floor)	Seepage ($m^3/m/s$)
7	Existing Zifta barrage section, no U/S clay, no D/S filter and CB, H = 5.5 m	0.1708	$8.16203 x10^{-5}$
	As above but H = 4.0 m	0.1243	$5.93602 x10^{-5}$
	As above but H = 3.0 m	0.0932	$4.45202 x10^{-5}$
8	Existing Zifta barrage section, U/S clay depth = 1.5 m and Length = 13.9, no D/S filter and CB, H = 5.5 m	0.1587	$7.64895 x10^{-5}$
	As above but H = 4.0 m	0.1154	$5.56287 x10^{-5}$
	As above but H = 3.0 m	0.0866	$4.17215 x10^{-5}$
9	Existing Zifta barrage section, U/S clay depth = 0.5 m and Length = 13.9, no D/S filter and CB, H = 5.5 m	0.1609	$7.73981 x10^{-5}$
	As above but H = 4.0 m	0.117	$5.62895 x10^{-5}$
	As above but H = 3.0 m	0.0878	$4.22171 x10^{-5}$
10	U/S clay layer existing but with depth = 0.5 m and Length = 27.8 m, no D/S filter and CB, H = 5.5 m	0.15	$7.2588 x10^{-5}$
	As above but H = 4.0 m	0.1094	$5.27913 x10^{-5}$
	As above but H = 3.0 m	0.082	$3.95935 x10^{-5}$
11	U/S clay layer existing but with depth = 0.5 m and Length = 41.7 m, no D/S filter and CB, H = 5.5 m	0.144	$6.93885 x10^{-5}$
	As above but H = 4.0 m	0.1047	$5.04644 x10^{-5}$
	As above but H = 3.0 m	0.0786	$3.78483 x10^{-5}$
12	Existing section, U/S clay (1.5 * 13.9 m), no D/S filter and CB, H = 4.0 m, permeability of clay layer = $2.5 x10^{-9}$	0.1148	$5.53236 x10^{-5}$
	As above but permeability of clay layer = $2.5 x10^{-8}$	0.1148	$5.53525 x10^{-5}$
	As above but permeability of clay layer = $2.5 x10^{-7}$	0.1154	$5.56287 x10^{-5}$
	As above but permeability of clay layer = $2.5 x10^{-6}$	0.1197	$5.75193 x10^{-5}$
	As above but permeability of clay layer = $2.5 x10^{-5}$	0.1277	$6.13734 x10^{-5}$

By examining the results shown in (Table 10) and (Fig. 6) a significant effect of the U/S clay layer on the water seepage and the exit gradient values is found. All models were done without the D/S filter and without the D/S concrete blocks. The first model (No. 7) was done without the U/S clay layer to obtain basic results for the behaviour of the existing barrage. According to the results and the limits shown in Table 4, the admissible water head difference can be estimated. Also the water head difference can be calculated according to Bligh theory and according to Lane theory. The results of all the calculations for the admissible water head difference on existing Zifta barrage are shown in (Table 11) to ease the comparison between all the methods. By examining the results shown in (Table 10) and (Fig. 6) a significant effect of the U/S clay layer on the water seepage and the exit gradient values is found. All models were done without the D/S filter and without the D/S concrete blocks. The first model (No. 7) was done without the U/S clay layer to obtain basic results for the behaviour of the existing barrage. According to the results and the limits shown in Table 4, the admissible water head difference can be estimated. Also the water head difference can be calculated according to Bligh theory and according to Lane theory. The results of all the calculations for the admissible water head difference on existing Zifta barrage are shown in (Table 11) to ease the comparison between all the methods.



FIGIURE (6) Exit Gradients from the Parametric Study for U/S Clay Layer

TABLE (11) Maximum Permissible Head Difference using Bligh, Lane & FEM Methods

Method	No U/S clay	U/S clay is 1.5*13.9 m	U/S clay is 0.5*13.9 m	U/S clay is 0.5*27.8 m	U/S clay is 0.5*41.7 m
Bligh ($c_{B,crit}$ =18)	4 m	-	-	-	-
Lane ($c_{L,crit}$ =8.5)	4.8 m	-	-	-	-
FEM (allowable gradient =0.17)	5.5 m	5.9 m	5.8 m	6.2 m	6.5m
FEM (allowable gradient =0.14)	4.5 m	4.85 m	4.8 m	5.12 m	5.35 m

From (Table 11) it appears that the thickness of the U/S clay layer does not affect the seepage significantly as reducing the thickness from 1.5 m to 0.5 m made only little difference in the results. Doubling the length of this layer to 27.8 m and increasing it three times to 41.7 m made strong effect and reduced the exit gradient values and hence increased the admissible water head difference to 5.12 m and 5.35 m instead of 4.8 m for the case of 13.9 m length U/S clay layer. For comparison, this water head difference is 4.5 m for the case of no U/S clay layer. These values are obtained by comparing the exit gradients with the values given in (Table 4). Both the lower and the higher limits were used, although the higher limits are recommended to be used for carrying out parametric analysis by assuming failure of any sheet piles or increasing the permeability of the soil.

The presented results indicate that implementation of an U/S clay layer for such structures is an effective method to rehabilitate the structures and to increase their maximum admissible water head difference and hence to increase the storage capacity.

Also the results show that a depth of 0.5 m for such clay layer is enough taking into consideration the construction using high quality control. A protecting material should be layered above this clay layer to guarantee that it will be permanent in its location, as in existing Zifta barrage. It will be more effective to increase the length of this layer to obtain higher increase in the admissible water head difference. The length and depth of this layer must be verified using the Finite Element Methods because the traditional Bligh and Lane methods are not capable to take such a clay layer into account.

Conclusion

The results given above confirm that both the Bligh and Lane methods can be used to obtain an initial impression of the necessary apron length. This is also in agreement with experience. For the final design it is, of course, necessary to take the real conditions into account, i.e. in particular to consider the location effect of sheet piles. This can be done by the finite-element method, which yields the maximum exit gradient for comparison with the admissible values given by Khosla (Novak 1992). The calculations show that the lower exit gradient limit should be used for normal conditions, but the higher limit can be used for extreme cases.

A strong influence of the factor of anisotropy of the sand on the maximum exit gradient was found. This indicates that thorough piezometer measurements U/S and D/S of such structures are required to calibrate the seepage model.

According to the calculations, the existing Assiut Barrage should be able to hold a water head difference of up to 4.60 m for a short period in order to store more water (in case of sudden rainfall in the north of Egypt). This recommendation must be verified by more investigations concerning structure analysis.

The U/S clay layer can be used as a rehabilitation option to reduce the seepage under the barrage and hence to reduce the exit gradient values and to increase the factor of safety against piping. The thickness of this layer must not be very high, because the main factor is the length. It is thus recommended to increase its length to obtain the best results. To guarantee that this U/S clay layer will be permanent and will not be washed with the flow it should be protected with U/S protection.

The D/S filter material has a great effect on the results of the seepage model and it attracts the seepage water under the barrage. If this filter has a high thickness, high gradients under the filter and low exit gradient D/S the filter are obtained. The filter should be designed to suit the geometric and hydraulic criteria as explained in the presented paper and it can be used as a rehabilitation option to increase the safety factor against piping.

Further investigation to Zifta barrage is required particularly to investigate the properties of the filter layer. Construction of piezometers D/S and U/S the barrage is recommended, for instance to estimate the exact local anisotropy factor for the soil underneath the barrage.

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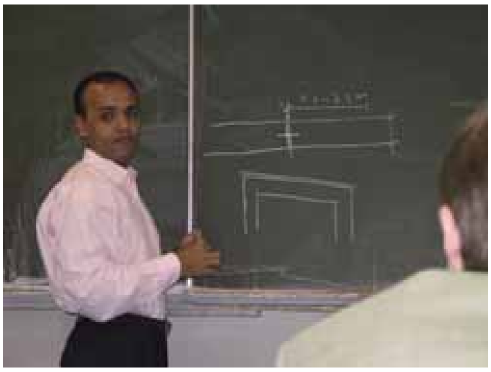
Analysis of Rigid Pavement with Non-Linear Unbound Granular Base under Traffic and Thermal Loads

Abstract



Consideration of non-linearity in unbound layers is necessary for accurate modelling of a flexible pavement structure. The necessity of non-linear consideration in rigid pavements has not been studied as closely. Many problems arise from the assumption of elastic layer properties for unbound layers. For example, granular layers may have a lower modulus than the sub-grade, and measured vertical strain at the top of the sub-grade may be twice the theoretical value.

The objective of this research is to analyse the rigid pavement with non-linear unbound granular base under traffic and thermal loading, with the goal to optimise the design criteria.

The analysis shall be conducted using the commercial finite element program ABAQUS available in the university of Hanover. The research shall be carried out through the following steps.

1- Investigation of the behaviour of the unbound granular materials to allow for proper modelling in FEM, many researchers have investigated that behaviour and found that, the unbound granular materials have mechanical properties that are stress dependant. It is found also that the rutting in flexible pavement happens due to rutting initiated in UGMs because of accumulated permanent strains under repetitions of traffic loads. This permanent strains have not yet considered in the analysis of the rigid pavements.

2 - Modelling of unbound granular materials according to the investigation mentioned above as a non-linear material in FEM through implementation of User-Subroutine to define the Constitutive Model for that material. Results from FEM analysis shall be compared to the experimental results to validate the constitutive laws implemented in the User-Subroutine.

3 - Modelling of rigid pavement in ABAQUS Program. A three dimensional model consists of three concrete panels each 4.25 x 4.25 m, thickness 0.30 m supported over an unbound granular base 0.60 m thickness with a sub-grade depth 3,00 m is chosen. The traffic loads are applied as static loads with load cases, middle of the panel and edge of the panel. A reasonable model for the rigid pavement is built in that way to take into account the following:
    - The interface between concrete panels through the dowels elements.
    - Sufficient geometry to define the boundary conditions in a better way.
    - The consideration of the moving traffic loads.
    - The consideration of thermal loads.
    - Combination between traffic and thermal loading.
    - The behaviour of concrete panels subjected to cyclic stresses arising from the above combination.

4- A parametric study for the optimisation of the pavement design in terms of concrete panel's thickness, unbound layer's thickness, properties of unbound layer material, and also the characteristics of the applied loads.

5- Conclusion of the study and recommendations to the practice design of rigid pavements to optimise the cost and life period of the pavement.

PILOTES
TERRATEST

RÜCKVERANKERTE GENEIGTE STÜTZWAND FÜR DEN VERKEHRSWEG “MALLECO”
***COLLIPULLI** NACHGESPANNTES PERMANENTANKERSYTEM*
UND SPRITZBETONWAND GROSSER MÄCHTIGKEIT

Aldo D. Guzmán G.

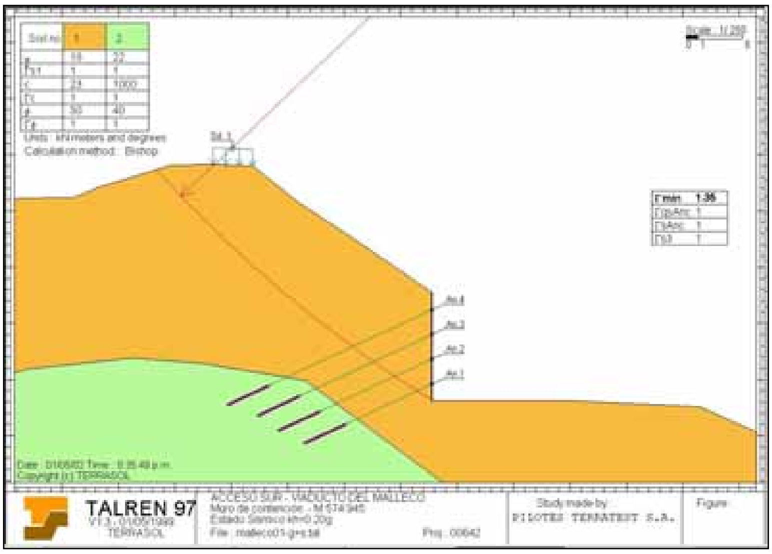
1. Einleitung

Das Projekt an der zweispurigen Verbindungsstrasse “Ruta 5 Sur” zwischen Collipulli und Temuco bestand aus der Verbreiterung bestehenden Strecke der Südtangente “Malleco”. Dieser Verkehrsweg befindet sich direkt südlich der Stadt Collipulli und zugleich an südlichen Zugang zur Autobahn an einem Hang starker Neigung, welcher in der Vergangenheit gravierende Probleme bei der Gewährleistung der globalen Standsicherheit mit sich brachte. 1973 wurden Hangbewegungen beobachtet und es wurden bereits zu dieser Zeit Verbesserungsmaßnahmen wie das Einbringen von Dränageleitungen und Pfählen am Hangfußpunkt notwendig. Nach diesen Maßnahmen wurden keine weiteren Probleme dieser Art am Hang beobachtet.
Voraussetzung für die Verbreiterung der bestehenden Fahrbahn der Strasse beinhaltete in der ursprünglichen Projektierung die Ausführung einer Auffüllung. Man folgerte allerdings, dass die eventuelle Konstruktion der besagten Auffüllung durch die Erhöhung des Eigengewichtes die Gesamtstandsicherheit der Böschung ungünstig beeinflussen würde.
Als alternative Lösung wählte man deshalb die Böschung weiter oben einzuschneiden, um die Verbreiterung der Fahrbahn zu ermöglichen. Dieser Vertikalschnitt erforderte den Rückhalt der Bodenmassen der Böschung und der Verkehrslasten der oberhalb

liegenden Bahnstrecke, welche das zentrale Tal Chiles von Norden nach Süden überquert.
Als Lösung wurden Verankerungswand und Bodenvernagelung als Stützbauwerke untersucht und es wurde sich aufgrund des Lastniveaus im anstehenden Projekt für die erste Möglichkeit entschieden.

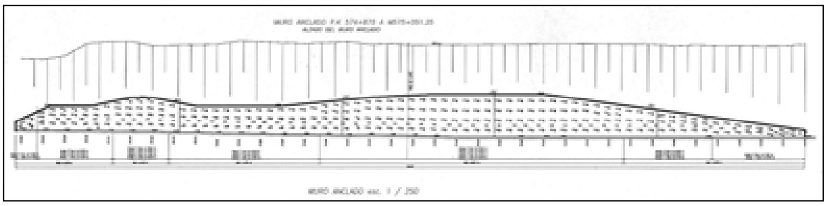
2. Planung und Geometrie

Anfangs bestand das Bauvorhaben aus einer Mauer von etwa 180m mit einer Höhe von 3 bis 10m und 7m im Mittel. Es existierten einige alte Sondierergebnisse und seismische Erkundungen, welchen ein relativ nahe der Mauer anstehender Fels zu entnehmen war. Man entschied sich für den Entwurf alle Anker in den Fels einbinden zu lassen, um Anker teils in den Boden, teils in den Fels einbindend zu vermeiden, was negative Effekte auf die Langzeitstandfestigkeit haben kann.
Der Entwurf für die Verankerung der Mauer wurde mittels des Programms TALREN realisiert, welches Beschleunigungen der Erdmassen durch Erdbeben mitberücksichtigen kann (vgl. Abb.).



Das Ergebnis des ersten Entwurfs, basierend auf der Annahme dass Fels gemäß dem seismischen Profil vorhanden sei, war eine Verankerung mit 239 Ankern mit Gebrauchslasten zwischen 700 kN und 825 kN und Längen zwischen 10 und 23,5 m.

Der Entwurf der bewehrten Betonmauer wurde mit einem strukturanalytischem Programm realisiert, das eine Modellierung des Untergrundes durch gleichmäßig verteilte elastische Federn erlaubt. Die Steifigkeit dieser Federn wurde abhängig von charakteristischen Bettungsmoduln abgeschätzt. Damit das Modell sich realitätsgetreu verhält, wurde zudem die Ankerkraft auf einen bestimmten Einflussbereich verteilt angenommen. Die Ausführung der Mauer aus Beton des Typs H30 wurde, in Übereinstimmung mit den Empfehlungen für Entwürfe nach ACI-318, zu einer Mächtigkeiten von 28 bis 30 cm abhängig von der Höhe und doppelt bewehrt ermittelt.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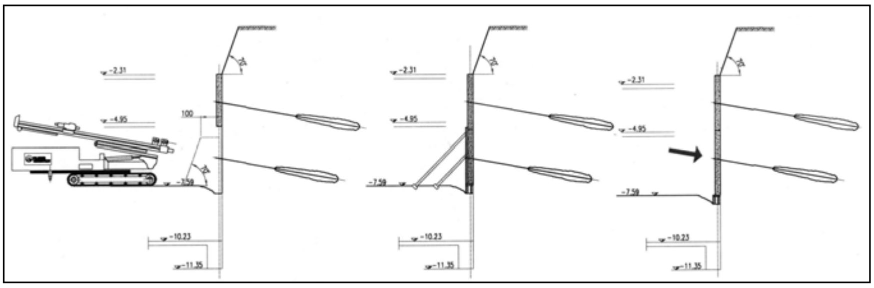
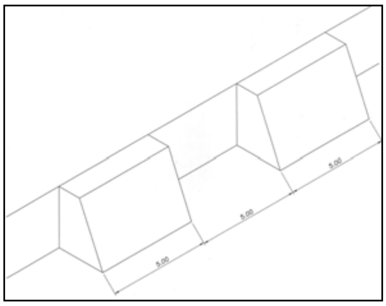
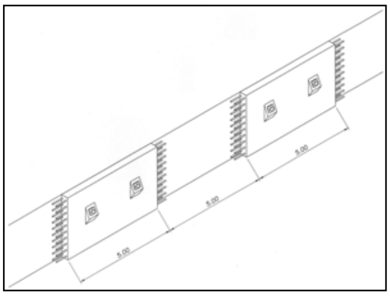
3. Konstruktion und Projektänderung

Die Mauerkonstruktion wurde auf die Art schrittweise mit Spritzbeton realisiert, dass bis die obere Auskleidung bereits eine zugesicherte Stabilität hatte, das angrenzende Erdreich noch nicht ausgehoben war oder durch bereits hergestellte Betonschalung gehalten wurde.



Der normale konstruktive Bauablauf bestand darin die Anker zu bohren, die Bewehrung zu installieren, dann den Spritzbeton aufzubringen und letztendlich die Anker zu spannen und über eine Injektion über den Ankerkopf zu sichern. Die Permanentanker mussten vor Ort, in Abhängigkeit von der notwendigen Länge gemäß den durchgeführten Bohrungen, hergestellt werden. Außerdem mussten die Anker Permanentanker sein oder doppelt gegen Korrosion geschützt werden. Das wurde durch ein Polyethylen-Rohr, welches den gesamten Kopfbereich ummantelt und damit schützt, realisiert.


5.00
5.00
5.00


5.00
5.00
5.00

Schon die ersten Ankerbohrungen zeigten, dass sich der Fels nicht in der gemäß seismischer Erkundung erwarteten Lage befand. Der Unterschied zwischen der erwarteten Position des Fels und der tatsächlich vorgefundenen Situation war so gravierend, dass es notwendig war die Anker derart neu zu bemessen, dass sie nun vollständig in den anstehenden tragfähigen Boden einbanden. Der tragfähige Baugrund war ein mitteldichter Schluff hoher Tragfähigkeit, der geotechnisch als Granodiorit V klassifiziert werden kann. Um die mögliche Traglast der Anker in dieser Schicht zu testen, und aufgrund der Tatsache, dass die neue Lösung ursprünglich nicht vorgesehen war, wurde es notwendig Ankerprobebelastungen durchzuführen, welche nach DIN 4125 bis zu Grenzlasten von 1270kN durchgeführt wurden.

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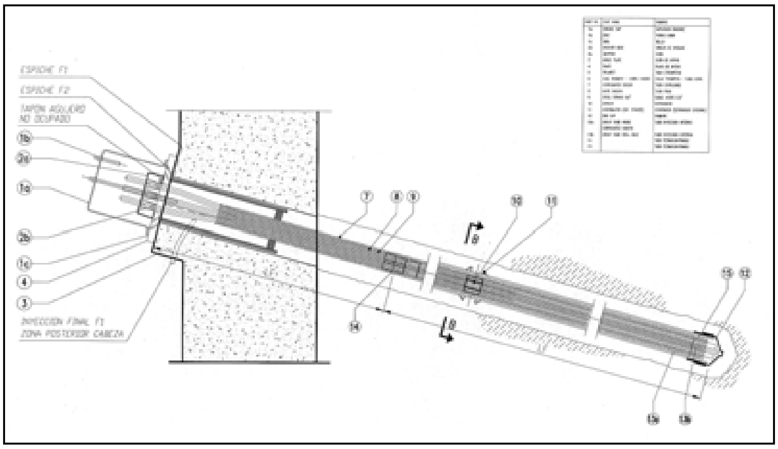
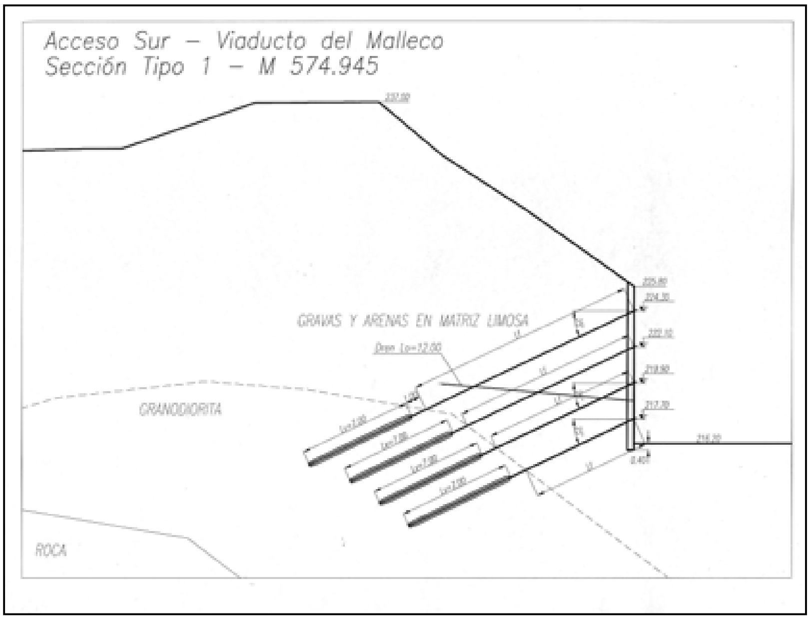

ESPICHE F1
ESPICHE F2
TAPON AGUJERO NO OCUPADO
INYECCION FINAL F1 ZONA POSTERIOR CABEZA


Acceso Sur – Viaducto del Malleco
Sección Tipo 1 – M 574.945
237.00
225.80
224.30
222.10
219.90
217.70
216.20
GRAVAS Y ARENAS EN MATRIZ LIMOSA
Dren Lo=12.00
GRANODIORITA
ROCA
Lv=7.00
0.40

Es wurden insgesamt 3 Probebelastungen nahe des Bereiches der geotechnischen Sondierungen durchgeführt, welche eine maximale Mantelreibung der Anker von 200 kN/m² für die Bemessung der Verpresskörper der Verankerung erlaubten. Das beobachtete Kriechen war in diesem Zeitraum kleiner als 0.8 mm, was damit kein maßgebendes Entwurfskriterium war. In Anlehnung an dieses Entwurf, wurden Verpresskörperlängen von 7m für Traglasten von 725 bis 825 kN geplant.

Abgesehen von dem Erfolg der durchgeführten Probebelastungen, führte das Einhalten eines adäquaten Sicherheitsniveaus für den Geländebruch der Böschung zu einer Erhöhung der Gesamtlänge der Anker. Die Länge des Verpresskörpers des Ankers wurde so gewählt, dass die globale Sicherheit gegeben war und die Einbindetiefe in den tragfähigen Schluff mindestens 1 bis 4,5 m gemäß Norm betrug.

Die längsten Anker erreichten eine Länge von 35m, während die Gesamtankerlänge für das Projekt von 2596m auf 6153m anstieg.

4. Bauüberwachung

Als Kontrollelement der Stabilität der Mauer “Malleco” wurden für das Projekt vier Vertikalschnitte (vier Anker pro Schnitt) für die Überwachung der Anker im Gebrauchszustand vorgesehen. Dafür wurden in den besagten Sektionen Lastaufnehmer, deren Signale über ein Kabel direkt in eine Überwachungsstation übertragen wurden, in der sich ein Datenlogger befand. Außerdem war in dem Gerät ein seismischer Sensor zur Beobachtung der Ankerlasten während eines Erdbebens installiert. Zudem wurden die Verformungen der Mauer in horizontaler und vertikaler Richtung topographisch dokumentiert.
Um andererseits die Gesamtstabilität des Hangs untersuchen zu können, wurden vier Inklinometerkanäle im Innern des Hanges untersuchen zu können,

wurden vier Inklinometerkanäle im Innern des Hanges eingerichtet, welche bei regelmäßiger Beobachtung erlauben das Gesamtverformungsverhalten des Hanges einzuschätzen.

Zusammenfassung

Anzahl der Permanentanker:	241
Gebrauchslast der Anker [kN]:	700-725-825
Bemessungskoeffizient für Erdbeben (kh):	0.20
Ankerlänge [m]:	15 a 35
Spritzbetonfläche [m²]:	1200
Mächtigkeit der Spritzbetonwand [cm]:	28-30

Aldo D. Guzmán G.
PILOTES TERRATEST S.A.
Alonso de Córdova 5151 - Of. 1401
Las Condes - CHILE
TE: ++56-2-437 2900
FAX: ++56-2-212 7235
www.terratest.cl

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KiKo
c/o AGTZE
Universität Hannover
Kommunikationssystem für
internationale Kooperation
Appelstrasse 9 A
D 30167 Hannover
☎ 0511 - 762-3370
🖷 0511 - 762-5105
mail@kiko-team.de

Antrag auf KiKo-Mitgliedschaft

Name: ______________________
Vorname: ______________________
Akad. Grad: ______________________
Strasse: ______________________
Ort ______________________
PLZ: ______________________
Heimatland: ______________________
Geburtsdatum: ______________________
Telefon: ______________________
Telefax: ______________________
e-Mail: ______________________
Beruf: ______________________

Sehr geehrte Damen und Herren des KiKo-Teams,

hiermit beantrage ich die KiKo-Mitgliedschaft und bin damit einverstanden, dass meine persönlichen Daten für diesen Zweck gespeichert und im Rahmen des KiKos verwendet werden.

....
Datum und Unterschrift

KiKo
c/o AGTZE
Universität Hannover
Kommunikationssystem für internationale Kooperation
Appelstrasse 9 A
D 30167 Hannover

☎ +49 (0)511 / 762-3370
🖷 +49 (0)511 / 762-5105

mail@kiko-team.de