MONITORING AND ANALYSIS OF SETTLEMENT AND STABILITY OF AN EMBANKMENT DAM CONSTRUCTED IN STAGES ON SOFT GROUND

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Abstract

A Probabilistic Three Dimensional Slope Stability Analysis (PTDSSA) Model is developed to evaluate the stability of embankment dams and their foundations under conditions of staged construction. The probabilistic 3-D slope stability analysis model is based on the following fundamental assumptions: (1) Failure surfaces are cylindrical; (2) The location and width of the sliding soil mass are constrained to their values; (3) the soil properties are statistically homogenous over the soil volume under consideration; (4) Cross sections along the axis of the slope are the same; (5) The uncertainty in the unit weight of soil and slope geometry is of negligible order compared to those associated with other soil parameters.

The PTDSSA mnodel is coded in a computer program capable of analyzing slopes located in multilayered deposits and under short and long-term conditions. The main input parameters are, the mean values and the uncertainties of the soil properties, average pore pressure and the associated variability and mean and coefficients of variation of different corrective factors and the correlation characteristics of the different soil parameters represented by the scales of fluctuation in the x,y and z directions.

The main outputs of the program are the geometric parameters of the most critical sliding surface (i.e. center of rotation/radius of rotation and critical width of failure), mean 2-D safety factor, mean 3-D safety factor, squared coefficient of variation of resisting moment and the probability of slope failure. The program is applied to a case study, karameh dam in Jordan. Monitored data of induced pore water pressure in the dam embankment and soft foundation were gathered during dam construction.

The stability of Karameh dam embankment and foundation was evaluated during staged construction using deterministic and probabilistic analysis. Foundation stability was evaluated based on the monitored data of pore water pressure.

The study showed that the mean values of the corrective factors which account for the discrepancies between the in situ and laboratory measured values of soil properties and for the modeling errors have significant influence on the 2-D safety factor, 3-D safety factor, slope probability of failure and on the expected failure width.

The degree of spatial correlation associated with shear strength parameters within a soil deposit also influence the probability of slope failure and the expected failure width. This correlation is quantified by scale of fluctuation. It is found that a larger scale of fluctuation gives an increase in the probability of slope failure and a reduction in the critical failure width.

Key Words: Embankment Dams, Shear Strength, Monitored Pore Pressure, Staged Construction, Uncertainity, Slope Stability, Factor of Safety, Probability of Failure.

INTRODUCTION

The evaluation of the stability of embankment dams and their foundations is always a major design consideration, which should take account of steady state seepage and rapid draw down conditions which could result in failure with catastrophic consequences. Construction on soft ground may generate positive excess pore water pressures within the underlying foundation soils, particularly during the construction phases. Ladd (1991) pointed out that appropriate drainage at each stage of the construction will enhance the strength of the most highly stressed soils and hence increase the safety factor against shear failure. Staged construction may involve either a continuous, controlled rate of loading or construction in two or more stages or a combination of both.

Three main sections along the dam were analyzed in details to evaluate the stability of dam embankment and foundation during staged construction, and to assess the capability of different soil mechanics models in predicting soil behavior under staged construction loading condition. Detailed sensitivity studies were carried out with respect to different input parameters.

The embankment and foundation of Karameh dam were found to be stable during construction.

METHODOLOGY FOR EVALUATING STABILITY OF KARAMEH DAM DURING STAGED CONSTRUCTION

The evaluation of the stability of Karameh Dam during staged construction was undertaken in the following steps:

1. Collection of Data on Karameh Dam:
   a. Geological information and geometry of the dam at the main cross sections (Chainages 800, 1480 and 1850).
   b. Geotechnical properties of the dam foundation (Atterberg limits, stress-strain and shear strength characteristics, consolidation and permeability characteristics, established in investigations between 1986 and 1996.
   c. Monitored values of pore pressure and settlement in the dam body and in the foundations.
   d. Schedule of construction stages and the levels of fill with time.
2. Evaluation of:
   a. Pore pressure using 1-D, 3-D and Skempton's methods.
   b. Back analysis of monitored field data to obtain a best estimate of field soil properties to compare with laboratory measured soil properties
   c. Evaluation of stability during staged construction using effective stress analysis (ESA) and undrained stress analysis (USA) using the computer program PCSTABL5M (Achilleos 1988).
   d. Probabilistic slope stability analysis during staged construction
3. Comparison between the predicted and monitored values in (2).
4. Development of guidelines for the most appropriate procedures to be followed for prediction and evaluation of the parameters related to dam performance during staged construction (i.e. pore water pressure, settlement and slope stability).

Reliability Analysis of the Stability of Embankments on Soft Clays

In reliability analysis, safety is determined by comparing the resistance against the applied load. In the case of slope stability analysis, M_R= resisting moment; M_D= Driving moment. When M_R is less than M_D, a shear failure occur.

If M_D and M_R are random variables then:

$$P_f = P_r(M_R < M_D)$$

where
P_f = Probability of failure

Then reliability of the system is the probability of survival = $1-P_f$.

However, M_D and M_R depend on other variables. For example, M_R depends on in situ soil properties and geometric parameters.

The resisting moment have uncertainty due to inherent spatial variability of soil properties (e.g. Vanmarcke, 1977 and 1980), uncertainty in the model used to obtain M_R and estimation errors for the soil parameters involved (Yucemen 1973, Yucemen et al. 1973, Yucemen and Al-Homoud, 1986 and Yucemen and Al-Homoud, 1988). These assumptions were adopted in this study.

Because of that, non-dimensional correction factors N_f and $N_{x_\imath}$ are introduced:

$$f = N\overline{F}$$
$$X_i = N(X_i)\overline{X}_i$$
$$M_R = f(X_1, X_2,.....X_n)$$

where
$\overline{F}$ = Theoretical function used to obtain M_R;
$\overline{X}$ = Random variable used to model X_i;
X_I = Any parameter involved in calculating the resisting moment.
N_f and $N_{x_\imath}$ = random variables with means $\overline{X}_i$, $\overline{N}_f$, $\overline{N}_{x_\imath}$ and coefficients of variations $\delta_{x_\imath}$, Ω_{Nf}, $\Delta_{x_\imath}$, respectively.

Then the total uncertainty inherent in any variable xi according to the first order, second moment (FOSM) method:

$$\Omega_{(xi)} = \sqrt{\delta^2_{(x_i)} + \Delta^2_{(x_i)}}$$

$\delta_{x_\imath}$ = Inherent variability associated with any of the basic variables x_i obtained from measured data, and

Δ_{x_i} = Is a measure for prediction uncertainty in x_i.

According to Yucemen et al (1973), N_{x_i} is a product of several corrective variables due to estimation errors of x_i:

$$N_{(x_i)} = \prod_{j=1}^{no} N_{j(x_i)}$$

where

$N_{j(x_i)}$ = Corrective factor to compensate for the prediction error in x_i due to the jth effect.

n_o = Total number of correction factors.

By assuming all corrective factors are mutually independent and using the FOSM, the expected values of $N_{(x_i)} = E(N_{x_i}) = \overline{N}_{(x_i)} = \prod_{j=1}^{no} \overline{N}_{j(x_i)}$ with coefficient of variation =

$$\Delta^2_{(x_i)} = \sum_{j=1}^{no} \Delta^2_{j(x_i)}$$

$\overline{N}_{x_i}$ and $\Delta^2_{x_i}$ can be obtained as elaborated by Al-Homoud (1985). Assuming all the variable are independent with Gaussian distribution and according to Al-Homoud (1985), the probability of failure P_f=

$P_f = 1-\phi(\beta)$

β= reliability index = $\dfrac{\mu_{M_R} - \mu_{M_D}}{\sqrt{\sigma^2_{M_R} + \sigma^2_{M_D}}}$

where

μ = Mean

= Variance

Φ = Stand for the standard normal distribution function such that:

$$\mu_{M_R} = \overline{N}_f \hat{f}(M_1, M_2, ...M_n)$$

where $M_i = \overline{N}_{(x_i)}$ xi

$$\sigma^2_{M_R} = \sum_{i=1}^{n} \left(\frac{\partial_{M_R}}{\partial_{M_i}}\right) \sigma_i^2$$

where ∂ stand for the partial derivation notation.

The uncertainty in driving moment σ_{M_D} is neglected for slopes where weight of the soil above sliding surface is an external load.

As a result of spatial averaging of soil parameters, a reduction in the standard deviation and variance occur, the reduction factor = r_u for standard deviation and r^2_u for the variance.

PROBABILISTIC MODEL FOR 3-D STABILITY ANALYSIS

Assumptions

1. Failure surface is cylindrical.
2. Location and width of sliding mass are at their critical values.
3. The soil properties are statistically homogenous over the soil volume
4. Cross sections along axis of the slope are the same.
5. Uncertainty in unit weight of soil and slope geometry is negligible.

Inherent Variability of Resisting Moment M_R

The randomness of M_R Resistance Moment described by its μ_{M_R} mean standard deviation and scale fluctuation, λ_{M_R} .

Those statistical parameters depend on the spatial average of the shear strength properties. Spatial average of shear strength differs from point value of shear strength. According to Al-Homoud (1985), if S and s are the point and spatial average values of the undrained shear strength, $\widetilde{S}$ is the standard deviation and $\widetilde{s} = r_s(L)\widetilde{S}$, where L = total arc length ; $L = \sum_{i-1}^{m} l_\iota$, $r_s(L)$ is the reduction factor (Yucemen and Al-Homoud, 1990).

Vanmarke (1977), suggested that the 3-D formulation will deviate from the 2-D formulation by taking into account the end effects. According to Vanmarke (1977), 3-D safety factor:

$$F_b(x_o) = \frac{M_{R,b}(x_o)}{M_{D,b}(x_o)} = \frac{M_{R,b}(x_o) + R_e}{M_{D,b}(x_o)}$$

R_e = Resisting Moment contributed the end sections = 2Ar's,
where
s = special average shear strength over the end sections,
A = Area of cross sectional are of sliding mass at the ends,
r' = Effective rotation arm for the end sections.
$M_{D,b}(x_o)$ Driving moment
$M_{R,b}(x_o)$ = Resisting moment

Probabilistic Three Dimensional Slope Stability Analysis Program (PTDSSA)
General Remarks

The PTDSSA computer program was originally developed by Al-Homoud (1985). This computer program is prepared to carryout the numerical computation associated with the probabilistic model. This program can analyze slopes located in multilayered deposits and under short and long-term conditions. The main input parameters are, the mean values and the uncertainties of the soil properties, average pore pressure and the associated variability and mean and coefficients of variation of different corrective factors and the correlation characteristics of the different soil parameters represented by the scales of fluctuation in the x,y and z directions (Table 1) (Yucemen and Al-Homoud, 1990).

The main outputs of the program are the geometric parameters of the most critical sliding surface (i.e. center of rotation/radius of rotation and critical width of failure), mean 2-D safety factor, mean 3-D safety factor, squared coefficient of variation of resisting moment and the probability of slope failure (Table 1) (Yucemen and Al-Homoud, 1990). The analysis of geotechnical incorporates uncertainties from several sources, uncertainties about site geology, and uncertainties about soil properties derive from sampling.

PTDSSA program is used here to evaluate the stability of Karameh dam during staged construction using Undrained Strength Analysis (USA).

CASE STUDY: KARAMEH EMBANKMENT DAM

Karameh dam is one of the major landfill dams in Jordan constructed on the Wadi Mallaha. The main purpose of the reservoir is the storage of the surplus winter flow in the King Abdullah canal to allow irrigation at the southern end of the valley.
The works included a 45 m high zoned earthfill dam (Table 2) with spillway and draw off structures.

Design and Construction of Dam Project

Construction Materials

The zoned embankment dam is constructed of the following materials.

Core zone- Middle clays and upper clays.
Shoulder zone - Sands and gravel from the dome of Samra formation material at the north end of the reservoir.- Alternatively-Lisan marl from the reservoir area.
Filter and transition zone - Wadi fan sands and gravels to the east the of reservoir area.

Drainage material - Gravel obtained by screening of wadi gravels.
Rock fill and riprap - Selected wadi fan gravels and/or quarry in escarpment limestone and sandstones.

Three typical dam sections were selected for the analysis in this study: Ch 800, Ch 1480 and Ch. 1850. For briefness, presentations are given in this paper for the dam cross section at Ch 1480.

Geotechnical Properties of Karameh Dam Foundation and Fill Properties

Stability evaluation of Karameh Dam during construction required the assessment of the pore water pressure at selected piezometers located in the foundation and within dam body.

The monitoring data are compared with the predictions using made soil mechanics models. Control samples were used to evaluate the accuracy of the monitored data and calibrate the prediction models including parametric studies. Based on the outcomes of this comprehensive exercise, the stability of the dam was evaluated during construction.

It is worth mentioning that all the above tests results were obtained from tests conducted in Jordan and the United Kingdom in professional labs using the state of the art testing equipment and following strictly the related ASTM standards.

To conclude on geotechnical properties, Tables 3 through 5 summarize the principal design parameters for the foundation and dam material. This includes density, effective cohesion and angle of friction, permeability, total friction and angle of friction, compressibility and consolidation coefficients, etc. Table 6 lists the values for the pore pressure parameter for the different materials used in dam construction and the foundation material as used by Sir Alexander Gibb (1988) in the design of the dam prior to any measurement of loading induced pore pressure during staged construction of the dam.

Furthermore, Table 7 gives the upper bound, lower bound and design values of undrained shear strength for the Lisan foundation material.

Pore Water Pressure Data

An analysis of monitored loading induced pore water pressure in the foundation, pore water pressure parameter (B-parameter) can be obtained as a ratio of loading induced pore water pressure to loading induced vertical stress for installed Piezometers.

To facilitate analysis of monitored data, and stability analysis of staged construction, the construction is divided into three stages.

Furthermore, to facilitate analysis related to stability analysis using the undrained shear strength obtained using the normalized shear strength relation given in the literature review.

Based on these stress results and the already available undrained shear strength test results obtained from laboratory experiments conducted on foundation material at specified effective stress and overconsolidation pressures. The normalized relation of undrained shear is used to obtain the variation of undrained shear strength with depth in the foundation at the three representative dam sections.

Based on monitored data of pore water pressure, the variation of measured pore pressure heads with height of fill and also with time were obtained for all piezometers installed in the foundation underneath the dam. Analysis of measured values of pore pressure show that the B-parameter in the foundation tends to decrease from its initial value (0.9) with height of fill and through construction time.

Based on the measured pore water pressure height, the pore pressure parameter r_u in the dam core was evaluated adopting a value of 18.0 t/m^3 for the unit weight of core material. The monitored data of pore pressure parameter r_u in fill are compared to those obtained using the prediction model.

Stability Analysis of Staged Construction

Since, Karameh dam was constructed on saturated, soft ground interbedded by lenses of sand layers, the construction was executed at stages. Since the construction generates positive excess pore water pressures within the underlying foundation soils, the most critical stability condition occurs during construction. However, drainage due to consolidation after each load application will progressively strengthen the stressed soils and hence increase the factor of safety against a shear induced failure. Such staged construction involve either a continuos, controlled rate of load application or construction in two or more stages, or a combination of both (Ladd, 1991).

The design of staged construction project must estimate both the initial strength of the cohesive soil and its rate of increase with time due to consolidation under the applied loads (Ladd, 1991).

The most important part of the field monitoring program for staged construction project concerns foundation stability. Limiting equilibrium stability analyses is used to obtain a quantitative measure of safety using field instrumentation data. The Effective Stress Analysis (ESA) could be used to check stability during construction based on measured pore pressures, whereas a Total Stress Analysis (TSA) can be used based on subsequent field vane testing of undisturbed sampling. Moreover, all accept the premise that effective stresses control the strength of soil (Ladd, 1991).

For stability analysis, the laboratory strength testing program included extensive triaxial tests and Oedometer test to obtain the profile of preconsolidation pressure and to measure the gain in undrained shear strength S_u with consolidation. For example The gain in undrained shear strength can be obtained using the normalized behavior technique (Ladd 1991) using the values of S and m (evaluated using triaxial CU tests) and the updated Overconsolidation Ratio (OCR) of foundation based on the initial OCR and the measured loading induced pore water pressure. Such normalized undrained strength relations are developed here for the foundation material along Karameh dam.

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The stability analysis of Karameh dam embankment and foundation is carried out using a deterministic approach The two-dimensional limiting equilibrium PCSTABL5M program (Achilleos, 1988) is used here to calculate the factor of safety against slope failure. This program is used for both effective stress analysis (ESA) and undrained strength analysis (USA).

PCSTABL5M program (Achilleos, 1988) is capable of calculating the factor of safety against slope instability using the simplified Bishop (Bishop, 1955) method of slices, which is applicable to circular shaped failure surfaces. PCSTABL5M (Achilleos, 1988) program incorporates also the simplified Janbu method (Janbu, 1954) of slices, which is applicable to failure surfaces of a general shape, or Spencers method of slices which is applicable to surfaces having a circular or general shape.

The most important field observations during construction of Karameh Dam were:

a) Additional fill placement in the central part of the dam would cause a rise in piezometric elevation.
b) At Ch 1850 the foundations behaved as predicted with low incremental B values.

c) The critical section with lowest factor of safety against failure was Ch 1480. The piezometers beneath the core, below the upstream 1:3 slope and beyond the berm junction, all indicate a much reduced rate of pore pressure increase with time. There is a correlation between the loading rate and the dissipation rate.

Probabilistic Stability Analysis of Karameh Dam

Average Soil Properties

Probabilistic stability analysis based on Undrained Strength Analysis (USA) is carried out for the main chainages of Karameh dam using PTDSSA program. As in the case of deterministic stability analysis, the same critical failure surfaces at Ch. 800, 1480, 1850 can be used here. Moreover, as in the case of deterministic stability analysis, the undrained shear strength values are determined for staged construction at main sections of Karameh dam based on monitored values of pore water pressure induced during loading using SHANSEP procedure (Ladd & Foott, 1974).

Estimation of Corrective Factors

The following values for the mean and c.o.v. of the corrective factors have been found based on previous studies by Yucemen and Al-Homoud (1990).

The Karameh dam clays have a low sensitivity. Hence the values of $N1(s_u)$ and Δ_1 (s_u) are given to be 1.00 and 0.117, respectively for the four materials given in Tables 8. For the discrepancy resulting from the mechanical disturbance during sampling $N_2(s_u)$ = 1.00 and Δ_2 (s_u) = 0.0, respectively for the four materials given in pervious Tables.

The values of N_3 (s_u) and Δ_3 (s_u) are given to be 1.15 and 0.08, respectively for the pervious four materials. By assuming the sensitivity to rate of shearing to be slightly sensitive, values of N_4 (s_u)= 0.93 and Δ_4 (s_u)= 0.05 are given for the four materials. For the effect of anisotropy N_5 (s_u)= 0.87 and Δ_5 (s_u)= 0.11. For the effect of progressive failure, N_6 (s_u)= 0.93 and Δ_6 (s_u)= 0.04, respectively for the four pervious foundation and fill materials.

Estimates of Scales of Fluctuation

No data are available to estimate the scales of fluctuation for the undrained shear strength, and the best estimate is found to be λs_u = 1.0 m for the four foundation and fill materials based in the study by Yucemen and Al-Homoud (1990).

Sensitivity Analysis

The best estimates of the different parameters involved in the stability analysis were used to calculate the mean value of the 2-D, 3-D safety factors, the probability of slope failure $P_f(B)$, the reliability Index (β), and the critical failure width b_c. Since the estimated values of these parameters could be in error, it is desirable to study the sensitivity of the above results to the variations in these parameters. For this purpose, one parameter will be perturbed within a certain range while the other parameters are fixed to their best estimates, and the variation in the basic results of the PTDSSA model like P_f (B), 2-D safety factor, b_c, etc. will be examined.

Parametric analysis are made for the dam sections at main chainages along Karameh dam. The parametric analysis include varying the model corrective factor N_f from 1.0 to 1.3, varying **(Ω_{Nf})** from 0.11 to 0.25, varying the corrective factors of progressive failure N_{sf} from 0.8 to 1.0, varying **(ΩN_{Sf})** from 0.04 to 0.3, varying total slope width B from 100 to 2000m, varying (λ_z) from 2 to 10 m, varying (λ_x) = (λ_y) from 2 to 10 m, and varying the coefficient of variation c.o.v's of undrained shear strength from 0.0 to 0.33. In the following, the results of this sensitivity study are presented.

Probabilistic approach is used here because there is uncertainaty in soil properties due to disturbance during sampling, size of specimen, rate of shearing, sample orientation and anisotropy and progressive failure.

The following are the effect of the sensitive parameter on 2-D safety factor, 3-D safety factor, probability of slope failure $p_f(B)$, reliability index, β, and critical width, b_c.

(a) Sensitivity to Corrective Factors

Introduction of corrective factors and the corresponding coefficients of variations are major contributors to the risk of failure, and affects the prediction of the failure surface, the following is the effect of the different corrective factors on the result:

Effect of N_f and Ω_{Nf}

As N_f changes from 1.0 to 1.30. It is observed that an increase of N_f from 1.0 to 1.30 will increase 2-D S.F from 1.36 to 1.77 at final stage of 40.5m fill at chainage 1480, increase 3-D safety factor from 1.88 to 2.45, and decrease $p_F(B)$ from 0.459 to 0.223. Moreover, the other stages of fill at Ch. 800, 1480 and 1850 behave the same trends.

An increase of Ω_{NF} from 0.0 to 0.25, increase $p_F(B)$ from 0.309 to 0.462. Moreover, the other stages of fill at Ch. 800, 1480 and 1850 behave the same trends. All results are quite sensitive to the estimated parameters of corrective factors accounting for the modeling error.

Effect of N_{Sf} and $\Omega(N_{Sf})$

Mean corrective factor N_{Sf} and the corresponding uncertainty $\Omega(N_{Sf})$ have similar effect on the results as those observed in the case of N_f and $\Omega(N_f)$, respectively. The effect of N_{sf} and $\Omega(N_{Sf})$ on 2-D safety factor, 3-D safety factor, $p_f(B)$ is similar to those for N_f and Ω_{Nf}, respectively.

The main conclusion related to these corrective factors is that, an increase in the mean of any corrective factor leads to an increase in the safety, decrease in probability of slope failure, and an increase in the reliability index B. Also it should be noted than an increase in c.o.v. of these corrective factors increases the slope failure probability and decrease the reliability indexs β.

(b) Sensitivity to Total Slope Width B

The total width of the slope is considered to be variable, in order to predict its effect on the critical failure width b_c and the probability of slope failure $p_f(B)$.

For the cases where the total slope width B is greater than the critical width of failure (i.e. $B>b_c$), it is found that b_c will be approximately a constant value. By increasing the total slope width from 100m to 2000m at final stages of 18.5m fill at Ch. 800, the critical failure width is increased from 50 to 70m only. Therefore, we may say that the critical failure width b_c is almost constant for different total slope widths provided that $B>b_c$.

It is found that as the total slope width increases from 100m to 2000m, the $p_f(B)$ increases from 0.133×10^{-1} to 0.352 at the final stage of 18.5m fill at Ch. 800, $p_f(B)$ increases from 0.22×10^{-1} to 0.529 at Ch. 1480, and increases from 0.269×10^{-1} to 0.591 at Ch. 1850. The other stages of fill at Ch. 800, 1480, and 1850, behave the same trends.

Therefore, slopes with smaller widths appear to be safer than those with larger widths. It should also be emphasized that the total slope width has no effect on the conventional 2-D safety factor. The results justify the assumption of a certain value for the total slope width (when it is not known) since this assumption will affect significantly only the probability of slope failure, which may also be used as a relative measure of slope safety. However, in case where the total slope widths are unknown, the failure probabilities can be expressed per unit axial width of slope.

(c) **Sensitivity to Soil Properties (S_u)**

Effect of Average Undrained Shear Strength

It is observed that, the 2-D, 3-D Safety factors and reliability index B increase, the probability of slope failure $p_f(B)$ decreases, and the critical failure width b_c increases as the average undrained shear strength s_u increases.

Effect of Scales of Fluctuation (λ_x), (λ_y), and (λ_z)

Since the analysis here is an undrained strength analysis, the fluctuation characteristics of soil properties S_u influence the uncertainties in slice resisting moment due to spatial averaging over slice lengths and due to the correlation between slice resisting moments.
It is found that by increasing the scale of fluctuation from 2m to 10m, $p_f(B)$ increases from 0.168 to 0.391 at final stage of 40.5 fill in Ch. 1480. Moreover, the other stages of fill construction behave the same trends.

It is found that by increasing S_u from 2m to 15m, the critical width b_c decreases from 168m to 84m then increases to constant 126m for the same case. It is found that when $\lambda_x=\lambda_y$, the result is almost the same for λ_z variation.

The main conclusion for this analysis is that the strong dependence of properties over a wide area (high λ values) tend to increase substantially the risk. This shows the necessity for careful study of the type and degree of correlation of soil properties in a soil deposit.

Effect of $\delta(s_u)$

The main factors controlling the randomness of undrained shear strength s_u are the natural heterogeneity of soils and the sampling methods. Both effects are reflected by the coefficient of variations $\delta(s_u)$. It is found that an increase in $\delta(s_u)$ from 0.0 to 0.33 will increase the slope failure probability from 0.098 to 0.309 and it will decrease the critical width of failure from 36m to 84m at the final stage at Ch. 1480. Moreover, the other stages of fill construction behave the same trends.

(d) Sensitivity to Geometrical Parameters

Effect of Slope Height H

It is found that by increasing the slope height from 12m to 20m in at 800, 2-D safety factor decreases from 2.07 to 1.88, 3-D safety factor decreases from 2.84 to 2.59, $p_F(B)$ increases from 0.0374 to 0.182.

(e) Other Sensitivity Analysis Results

It is found that increasing the height of fill will decrease the 2-D and 3-D safety factor. Increasing the fill height from 30m to 40.5m, 2-D safety factor decreases from 1.92 to 1.58 and 3-D safety factor decreases from 2.73 to 2.18 at Ch. 1480. Moreover, the same trends at ch. 800 and 1850.

Ch. 1480 has the lower safety factor 1.58, Ch. 1850 has 1.65, then Ch. 800 has 1.88 2-D safety factor.

Results show that an increase in 2-D & 3-D safety factors and reliability index will decrease the probability of slope failure at the main sections along Karameh dam. For a given risk $p_F(B)$, one can find the required mean 2-D safety factor, 3-D safety factor and reliability index to be used for design purpose.

The probability of slope failure tends to increase with time until reaches a maximum values 0.182, 0.309, and 0.357 at end of construction of Ch. 800, 1480 and 1850, respectively.

SUMMARY AND CONCLUSIONS

The overall stability of Karameh dam embankment and foundation were evaluated using ESA and TAS methods. The monitored data were compared to those obtained using soil mechanics prediction models. Three main sections along Karameh dam were analysed in detail and it was concluded that:

1. The critical section along Karameh dam was at Ch 1480. It was important to monitor the actual performance in order to make appropriate design alterations during construction. The piezometers beneath the core, below the upstream 1:3 slope and beyond the berm junction, all indicated a much reduced rate of pore pressure increase with time. There was a good balance between the loading rate and the dissipation rate and no instability problems were encountered when in raising the fill. The othercross-sections analysed (Ch 1850, 1210 and 800) were also satisfactory.

2. Slope stability analysis safety factors evaluated using undrained strength analysis (USA) predicted lower safety factors than those evaluated using effective Stress Analysis (ESA). The

embankment and foundation of the Karameh Dam were stable during construction with low loading induced pore water pressures. Slope stability analysis gave a minimum safety factor of 1.31 and 1.22 using ESA, and USA methods respectively.

3. Undrained analysis using real lower bound strengths adjusted to the staged construction gave conservative factors of safety factor.

4. Pore pressure measurement allowed a better understanding of how the foundation was responding to loading such that it was unnecessary to require all the delays included in the contract.

5. In the short term stability, main sources of uncertainties are those associated with the method of analysis and the insitu value of undrained shear strength and in the long-term stability, main sources of uncertainties are those associated with the method of analysis, pore water pressure distribution, and the in situ values of angle of friction and cohesion.

6. The mean values of the corrective factors which account for the discrepancies between the insitu and laboratory –measured values of soil properties and for the modeling errors have significant influence on the 2-D safety factor, 3-D safety factor, slope probability of failure and on the expected failure width. It is found that a larger values of corrective factors increase 2-D safety factors, 3-D safety factors, and decrease the probability of slope failure. It is found that a larger values of c.o.v in corrective factors increase the probability of slope failure.

7. The degree of spatial correlation associated with shear strength parameters within a soil deposit also influence the probability of slope failure and the expected failure width. This correlation is quantified by scale of fluctuation. It is found that a larger scale of fluctuation gives an increase in the probability of slope failure and a reduction in the critical failure width.

8. Probabilistic three dimensional slope stability analysis influences the values of 2-D and 3-D safety factor, probability of slope failure reliability index and the critical failure width.

9. Total slope width has an important effect on the probability of slope failure, while it has not affected the critical failure width, significantly. It is found that a larger values of slope width increases the probability of slope failure.

10. For this case constructed on soft ground, it is concluded that the embankment and foundation of Karameh dam are stable during construction. Moreover, during construction, Karameh dam embankment and foundations are stable against slope stability. At the critical section, Ch 1480, a minimum safety factor of slope failure was 1.31 using ESA, 1.22 using USA and 1.33 using USA combined with probabilistic analysis. The maximum probability of slope failure was 0.59.

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REFERENCES

1. Achilleos E, 1988, User Guide for PCSTABL5M - Joint Highway Research Project, Report JHRP-88/19, School of Civil Engineering, Purdue University, West Lafayette, In., USA.
2. Ang A. H. S., and Tang W.H., 1975, “Probability Concepts in Engineering Planning and Design”, Volume I: Basic Principles, John Wiley and Sons.
3. Al-Homoud, A. S, 1985, “Probabilistic Three Dimensional Stability Analysis of Slopes”, M.Sc. Thesis, Dept. Civil Eng., Yarmouk University, Irbid, Jordan, pp. 390.
4. Bishop, A. W. 1955, "The Use of the Slip Circle in the Stability Analysis of Slopes, "Geotechnique, Vol. 5, No. 1.
5. Harr, M., 1977, “Mechanics of Particulate Media”, McGraw-Hill Books Co., Inc., New York, N.Y.
6. Holtz, R.D., and R.J. Krizek, 1971, “Statistical Evaluation of Soils Tests Data”, Proceedings, International Conference on Application of Statistics and Probability to Soil and Structural Engineering, Hong Kong.
7. Janbu, N., 1954, "Stability Analysis of Slopes with Dimensionless Parameters", Harvard Soil Mechanics Series, No. 46, 81 pp.
8. Ladd, C. C. and Foot, R., 1974, “New Design Procedure for Stability of Soft Clays”, Journal of Geotechnical Engineering Division, ASCE, Vol. 100, No.GT7, pp. 763-786.
9. Ladd, C.C., 1991, "Stability Evaluation During Staged Construction", Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 4, pp. 540-615, April.
10. Lambe, T.W., 1973, “Predictions in Soil Engineering”, Geotechnique Vol. 23, No. 2, pp. 149-202.
11. Lambe, T. W. and Whitman, R. V., 1979, "Soil Mechanics", John Wiley and Sons, New York.
12. Sir Alexander Gibb and Partners, 1986a, "Summary Report, Karameh Dam Project”, Vol. 1, April, Amman, Jordan
13. Sir Alexander Gibb and Partners, 1986b, "Preliminary Design for the Karameh Dam Project”, Karameh Dam Project, Vol. 3, April, Amman, Jordan.
14. Sir Alexander Gibb and Partners, 1986c, "Ground Investigations for the Karameh Dam”, Karameh Dam Project, Vol. 5, April, Amman, Jordan.
15. Sir Alexander Gibb and Partners, 1988, "Preliminary Design Calculations for the Karameh Dam, Karameh Dam Project, Final Report (Appendices), Vol. 7, April, Amman, Jordan.
16. Sir Alexander Gibb and Partners, 1993, "Storage Facilities in the Wadi Mallaha Karameh Dam Project, Phase B Supplementary", Design Calculation, Vol. 1 of 2, May, Amman, Jordan.
17. Sir Alexander Gibb and Partners, 1996, “Review Board Meeting 11th”, May, Briefing Document, Karameh Dam Project.
18. Sir Alexander Gibb and Partners, 1997, Report on Site Visit by project Manager, Karameh Dam Project, March.
19. Skempton, A. W. and Bjerrum, L., 1957, "A Contribution to the Settlement Analysis of foundations on Clay", Geotechnique, Vol. 7, p. 168.

20. Terzaghi, K., Peck, R. B., and Mesri, G., 1996, "Soil Mechanics in Engineering Practice", John Wiley and Sons, New York.
21. Terzaghi, K., 1943, "Theoretical Soil Mechanics", John Wiley and Sons, New York.
22. Vanmarcke, E.H., 1977, "Reliability of Earth Slopes", Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, GT11.
23. Yucemen, M. S. and Al-Homoud, A. S., 1986, "Safety of Earth Slopes: A Probabilistic Approach", Proceedings, IABSE Symposium on Safety and Quality Assurance of Civil Eng. Structures, Tokyo, Japan, pp. 125-132.
24. Yucemen, M. S. and Al-Homoud, A. S., 1988, "A Probabilistic 3-D Short Term Slope Stability Model", Proc. 5th ASCE Speciality Conf. On Probabilistic Methods in Civil Eng., Blacksburg, VA, pp. 140-143.
25. Yucemen, M. S. and Al-Homoud, A. S., 1990, "Probabilistic Three Dimensional Stability Analysis of Slopes", Structural Safety, 12, Elsevier Science Publisher B. V., Amsterdam, pp. 1-20

Key of Notation

Symbols used in the text are listed below:

H	Thickness of clay layer
$\sigma_{p'}$	Maximum past effective vertical stress/pressure
$\sigma_{vo'}$	Effective overburden pressure
cv	Coefficient of rate of vertical consolidation
t	Time
k_v	Vertical permeability
γ_w	Unit weight of water
B	width of loaded area
SF	Factor of safety
Δu	Excess pore water pressure
$\sigma_{vu'}$	Effective stress at end of loading
ESA	Effective stress analysis
TSA	Total stress analysis
USA	Undrained strength analysis
c' & ϕ'	Mohr-Coulomb effective stress failure envelope
S_u	Undrained shear strength
r_u	Pore pressure parameter
b	Critical Slope Width
c	Soil Cohesion
F_b	3-D Slope Stability Safety Factor
H	Soil Layer Depth
L	Slope Width
M_R	Resisting Moment
M_D	Driving Moment
N_f	Correction Factor for Modeling Error
P_f	Probability of Failure
r	Rotation Arm

Re	Contribution of End Side Section
rd	Stress Reduction Factor
r_u	Pore Pressure Reduction Factor
$r_u(x_i)$	Reduction Factor Resulting from Sptial Averaging of Soil Parameter xi
S_u	Undrained Shear Strength
$\overline{SF}$	Static Safety Factor
$\overline{SF}'$	Dynamic Safety Factor
S_q	Driving Force
ρ	Soil Density
τ_{av}	Average Shear Strength
σ_o'	Effective Overburden Stress
σ_o	Total Overburden Stress
β	Slope Angle
φ	Angle of Internal Friction
$\Omega(x_i)$	Coefficient of Variation of a Property xi
ρ_{xy}	Correlation Coefficient between Two Properties x, and y
Δ_{xi}	Coefficient of Variation of a Property xi Due to the Limited Number of Samples
Ω_m	Number of Maxima above $A_{r,b}$ by u(x,t)
ν	Upcrossing Rate
$\phi(\ )$	Gaussian or Normal Distribution Function

Table 1 The Input and Output Parameters of PTDSSA Analysis

Input	Output
- Slope Geometry - Layer Soil Properties: 1. Means, C.O.V's and Scale of Fluctuation of the Soil Properties, 2. Average Water Table and C.O.V. of Pore Water Pressure, 3. Mean Corrective Factors and the Corresponding C.O.V's. - Number of Slices - Trial Centers	- Geometric Parameters of the Most Critical Sliding Surface: 1. Center of Rotation 2. Radius of Rotation 3. Critical Width of Failure - Mean 2-D and 3-D Safety Factors - Squared C.O.V of the Resisting Moment - Probability of Slope Failure (Guassian and Beta Distributions).

Table 2 Summary of Karameh Dam Project Embankment Technical Data (Sir Alexander Gibb and Partners, 1997).

(a) General

The reservoir storage volume	55 M.C.M
The lake area	5.0 Sq. km
The lake length	4.8 km

(b) Dam body

Dam height (Maximum)	45 m
Dam length (approx.)	2050 m
Crest Width	10.00m
Volume of fill (Approx.)	11.2 Million Cubic Meter
Volume of excavation (Approx.)	6.4 Million Cubic Meter.

Table 3 Shear Strength Test Results and Density for Fill Materials (Sir Alexander Gibb and Partner, 1993).

Zone	Effective Stress		Total Stress		Density	
	c' (kPa)	ϕ' (Degree)	S_u * (kPa)	Su** (kPa)	Bulk (t/m^3)	Saturated (t/m^3)
1A	5	29	40	75	1.8	1.80
1B	5	29	80	100	1.87	1.96
2	0	38	-		2.1	2.20
3A	0	38	-		2.1	2.20
3B	0	38	-		2.1	2.20
4	0	38	-		2.1	2.20
5	0	38	-		2.1	2.20
6A	0	38	-		2.1	2.20
6B	0	38	-		2.1	2.20
7A	5	30	16		1.59	1.70
7B	5	30	80	100	1.82	1.91
7C	10	35	200		1.80	2.10
9	-	-	16		1.80	1.80

c' = Effective Cohesion, ϕ' = Effective Angle of Friction, S_u = Undrained Shear Strength.
* Lower Bound Derived from Consolidation Undrained (CU) Test
** Upper Bound

Table 4 Summary of Principal Design Parameters of Embankment Material (Sir Alexander Gibb and Partners, 1986b).

Material	Density		Cohesion*	Angle of Friction
	Bulk (t/m^3)	Saturated (t/m^3)	C'(kN/m^2)	ϕ'(0)
Foundations				
Middle Clay	1.93	1.93	12	32
Lower Laminated Unit	1.64	1.64	0	33
Lower Clay	1.81	1.81	15	25
Sand	2.0	2.0	0	35
Embankment Fill				
Core Zone	1.89	1.94	5	26
Shoulder Zone	1.99	2.09	0	35
Filters, Drains, etc..	2.04	2.14	0	35

* Effective Values

Table 5 Summary of Principal Design Parameters of Foundation Material (Sir Alexander Gibb and Partners, 1986b).

Unit Name	K avg. (m/s)	Cohesion (Kpa)	ϕ (Degree)	Mv (Mpa^{-1})	Cv (m^2/yr)
Main Laminated Unit	$1*10^{-8}$	10	35	0.35	150
Middle Clays	$1*10^{-10}$	12	32	0.33	2
Lower Clays	$8*10^{-11}$	15	30	0.31	2

Table 6 Pore Pressure Parameters for Fill and Foundation Material (Sir Alexander Gibb and Partners, 1988).

Zone	ru	$\overline{B}$
1A	0.7	-
1B	0.5	-
7A	0.7	-
7B	0.5	-
Foundation		0.7

Table 7 Undrained Shear Strength in Foundation Material (Sir Alexander Gibb and Partners, 1993).

Unit	Shear Strength (kN/m^2)			
	Lower Bound	Design Value	Upper	Expected
ML	30	55	180	60
MC	30	70	200	100
LL	65	100	400	150
Samra	130	175	-	-