VOL. 103 NO. GT8. AUG. 1977

JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

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DISCUSSION

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INFORMATION RETRIEVAL

The key words, abstract, and reference "cards" for each article in this Journal represent part of the ASCE participation in the EJC information retrieval plan. The retrieval data are placed herein so that each can be cut out, placed on a 3 × 5 card and given an accession number for the user's file. The accession number is then entered on key word cards so that the user can subsequently match key words to choose the articles he wishes. Details of this program were given in an August, 1962 article in CIVIL ENGINEERING, reprints of which are available on request to ASCE headquarters.

[a]Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

13129 ACOUSTIC EMISSION OF COHESIVE SOILS

KEY WORDS: **Acoustic detection; Acoustics; Clays;** Deformation; Microseisms; **Monitoring;** Nondestructive tests; Research; **Silts;** Soil mechanics; **Strains;** Stresses; Stress waves; **Triaxial tests**

ABSTRACT: In this, the second of a series of three papers on acoustic emission monitoring of soils, cohesive soils are evaluated. A study of acoustic emission fundamentals showed that the amplitude of the emissions in cohesive soils are from ½ to 1/400 of that in granular soils, that frequencies are in the low kiloherz range (predominantly 2 kHz-3 kHz), and that attenuation is high, from 30 dB/ft to 57 dB/ft (1.0 dB/cm to 1.9 dB/cm), and is related to water content. Regarding the macroscopic behavior of acoustic emissions in cohesive soils, the study showed that acoustic emission counts and strain are interrelated, both bearing a definite relationship to the imposed stress level, that increasing water content or plasticity index, or both, decreases the level of acoustic emissions generated, and that stress history effects are reflected in a change of acoustic emission rates. This latter finding allows for acoustic emission monitoring of undisturbed samples for the potential location of the preconsolidation pressure.

REFERENCE: Koerner, Robert M., Lord, Arthur E., Jr., and McCabe, W. Martin, "Acoustic Emission Behavior of Cohesive Soils," *Journal of the Geotechnical Engineering Division,* ASCE, Vol. 103, No. GT8, **Proc. Paper 13129**, August, 1977, pp. 837-850

13157 SHEAR STRENGTH OF GRANULAR MATERIALS

KEY WORDS: Assembling; Friction; **Friction factor;** Granular materials; **Internal friction;** Mechanics; **Shear strength;** Soil mechanics; **Statistical analysis; Triaxial tests**

ABSTRACT: Based on statistical methods, a theory has been proposed for the shear strength of a random assembly of spherical cohesionless particles. The theoretical analysis gives a relationship between angle of internal friction ϕ_{cv} at constant volume, interparticle friction angle ϕ_{μ}, and void ratio e. For comparison, drained triaxial compression tests have been carried out on steel spheres, uniform sands, and glass ballotini of different sizes. When ϕ_{μ} is calculated from the experimentally obtained ϕ_{cv} values it has been found that ϕ_{μ} is independent of void ratio, which is an indirect evidence to support the validity of the proposed theory. Comparison with experimental results on sands suggests that the use of the proposed theory could also be extended to random assemblies of irregular shaped particles as well.

REFERENCE: Sadasivan, Sekanoor K., and Raju, Vegesna S., "Theory for Shear Strength of Granular Materials," *Journal of the Geotechnical Engineering Division,* ASCE, Vol. 103, No. GT8, **Proc. Paper 13157**, August, 1977, pp. 851-861

13158 DESIGN OF MACHINE FOUNDATIONS ON PILES

KEY WORDS: **Cantilevers;** Design; Dynamics; **Dynamic structural analysis; Machine foundations;** Pile foundations; **Soil-structure interaction**

ABSTRACT: A practical method for the analysis of pile-supported foundations subjected to dynamic loadings is described. The method uses a single-degree-of-freedom mass-spring-dashpot model of a form similar to that used for shallow foundations to analyze the response of the actual system. The method of analysis uses the concept of an equivalent cantilever, which is a technique to simplify the soil-structure interaction problem, and allow the computation of equivalent spring constants in all modes of vibration. The method is demonstrated with an actual case history where computed and observed motions are compared.

REFERENCE: Singh, Jogeshwar P., Donovan, Neville C., and Jobsis, Adrianus C., "Design of Machine Foundations on Piles," *Journal of the Geotechnical Engineering Division,* ASCE, Vol. 103, No. GT8, **Proc. Paper 13158**, August, 1977, pp. 863-877

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ANALYSIS OF CONSOLIDATION BEHAVIOR OF MICA DAM

By Zdeněk Eisenstein,[1] M. ASCE and Steven T. C. Law,[2] A. M. ASCE

INTRODUCTION

An analysis of the construction behavior of an earth dam with a clay core, or more generally, of any embankment containing slowly draining material includes the problem of time dependency due to simultaneous generation and dissipation of pore pressures. Since pore pressures are known to influence the development of both total and effective stress fields within earth structures, such pressures must be considered if an overall stress and deformation analysis is attempted. The effects of pore pressures and their history are important not only for evaluating time-dependent components of deformation, but also for studying the phenomenon of stress transfer occurring due to the nonhomogeneous composition of the embankment. A number of published studies have demonstrated that any meaningful analysis of the behavior of earth dams during construction cannot ignore factors such as the nonlinear character of stress-strain relationships, the effect of incremental loading, and the influence of stress path followed during the loading process (e.g., 4,5,6,8,9,16,19,20). Thus, pore pressure analysis must recognize and be integrated with these factors.

Theories for estimating pore pressures generated in an earth dam during construction have been developed by a number of authors. One of the more recent methods of analysis (14) solved the problem using the theory of consolidation for two-dimensional drainage and a moving upper boundary. In this solution, the total stresses that generated the pore pressures were assumed equal to overburden pressures. However, due to the existence of stress transfer, vertical stresses in the core may differ considerably from overburden pressures

Note.—Discussion open until January 1, 1978. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 103, No. GT8, August, 1977. Manuscript was submitted for review for possible publication on April 1, 1976.

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(22,23). Thus in some cases the actual stress changes in the core should be taken into account.

In order to maintain strain compatibility, total stresses within a soil mass change during the consolidation process, even without a change in external load. Recognition of this phenomenon, implicit in the Biot (1) theory of consolidation, has been followed recently by finite element solutions (e.g. 12,13,21). Their limitations, at least from the point of view of an earth dam analysis, are that they do not readily incorporate nonlinear stress-strain relationships and do not consider incremental loading.

A new approach has been developed to make pore pressure analysis an integral part of a finite element analysis of stresses and deformations and to avoid limitations of the solutions cited previously. In this approach, a nonlinear incremental finite element analysis is combined with a finite element analysis of pore pressures. Both immediate (undrained) and time-dependent (drained) deformations are obtained, based on two sets (drained and undrained) of stress-strain data. This paper describes briefly the theoretical concept of this approach and examines it on the basis of a case history of consolidation behavior of Mica Dam in Canada. A more detailed description of the theoretical concept of this approach, consideration of the assumptions used in it and a parametric study of its sensibility has been presented elsewhere (7).

Finite Element Pore Pressure Analysis

Some of the previously developed analytical methods for estimating construction pore pressures in cores of earth dams solved the governing differential equations by the finite difference method (11,14). In order to combine the pore pressure analysis with a finite element analysis of stresses and movements in the entire dam, it becomes apparent that a finite element formulation of the consolidation problem would be superior to the finite difference method. Assuming that the strain condition prevailing within the centrally located core of a dam with stiff shells is one-dimensional confined compression, it can be shown that, for isotropic permeability, the two-dimensional consolidation with a moving upper boundary is described by the governing differential equation (10):

$$C_v\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)=\frac{\partial u}{\partial t}-\bar{B}\frac{\partial \sigma_1}{\partial t} \quad \ldots \ldots \ldots \ldots \ldots (1)$$

in which C_v = coefficient of consolidation of embankment material; u = pore pressure; σ_1 = total major principal stress for which changes occur only due to changes in applied loads; $\bar{B}$ = pore pressure coefficient; and t = time variable.

The pore pressure coefficient, $\bar{B}$, as introduced by Bishop (2) is a function of the principal stress ratio. It can be reasonably assumed that the central core in an earth-filled dam is sufficiently laterally confined to have a nearly constant total principal stress ratio. Thus the $\bar{B}$ values used in the analysis can be determined directly from laterally confined (oedometer) compression tests. Similar reasoning can be used to justify the use of oedometer tests to determine values of the coefficient of consolidation, C_v. The numerical technique used in solving the consolidation equation followed the finite element formulation of the heat conduction problem by Wilson and Nickell (26), which is described

by a differential equation of the same type as Eq. 1. Details of the computer program are given by Krishnayya (15).

The main advantages of the finite element formulation are: (1) The values of soil properties $\bar{B}$ and C_v (controlling the generation and dissipation of pore pressure) can be changed for any element at any stage of the construction

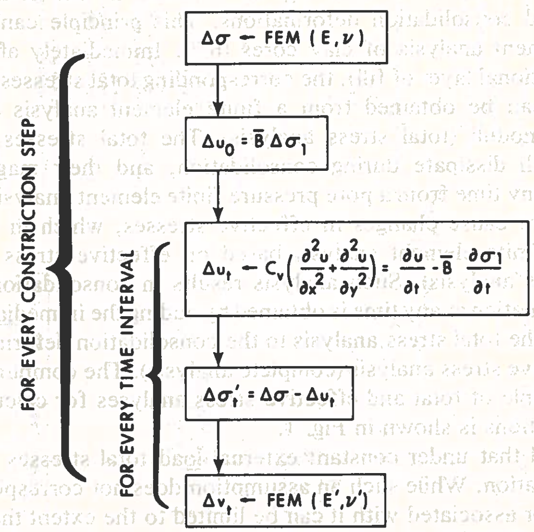


FIG. 1.—Scheme of Analysis of Consolidation Deformations

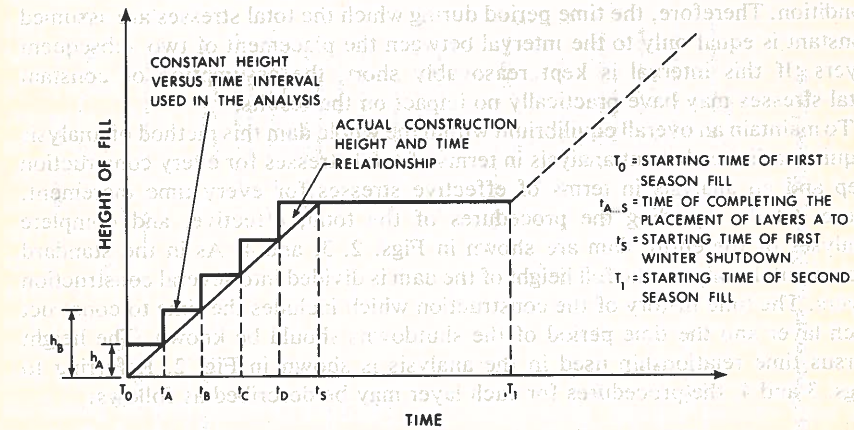


FIG. 2.—Height of Fill Versus Time Relationship as Used in Analysis

process depending on the state of stress or consolidation ratio achieved; (2) simplifying assumptions concerning the magnitude of the major principal stress are not necessary since the stress is obtained from the finite element total stress analysis, with which the consolidation analysis would be integrated; and

(3) any geometry or drainage boundary conditions and nonhomogeneous material distribution are easily modeled.

Total and Effective Stress Analysis

Deformations in cohesive soils can be obtained as the sum of immediate (undrained) and consolidation deformations. This principle can be introduced into finite element analysis of clay cores (6,7). Immediately after loading (or placing an additional layer of fill), the corresponding total stresses and immediate deformations can be obtained from a finite element analysis based on total stress elastic moduli (total stress analysis). The total stresses generate pore pressures which dissipate during consolidation, and their magnitude can be determined at any time from a pore pressure finite element analysis. The changes in pore pressure cause changes in effective stresses, which in turn constitute loading for a finite element analysis based on effective stress elastic moduli (effective stress analysis). Such analysis results in consolidation deformation. The final deformation at any time is obtained by adding the immediate deformation obtained from the total stress analysis to the consolidation deformation obtained from the effective stress analysis (complete analysis). The computational scheme using the principle of total and effective stress analyses for calculating consolidation deformations is shown in Fig. 1.

It is assumed that under constant external load total stresses do not change during consolidation. While such an assumption does not correspond to physical reality, the error associated with it can be limited to the extent that the practical results are not affected. Simultaneously with the consolidation process in the core, filling of the dam continues. This is analytically modeled by adding new layers to the dam, each representing a new loading and thus a new total stress condition. Therefore, the time period during which the total stresses are assumed constant is equal only to the interval between the placement of two subsequent layers. If this interval is kept reasonably short, the assumption of constant total stresses may have practically no impact on the results.

To maintain an overall equilibrium within the whole dam this method of analysis requires a finite element analysis in terms of total stresses for every construction step and an analysis in terms of effective stresses for every time increment. More details regarding the procedures of the total, effective, and complete analyses of the entire dam are shown in Figs. 2, 3, and 4. As in the standard incremental analysis, the full height of the dam is divided into several construction layers. The time history of the construction which includes the time to construct each layer and the time period of the shutdowns should be known. The height versus time relationship used in the analysis is shown in Fig. 2. Referring to Figs. 3 and 4, the procedures for each layer may be described as follows:

1. A finite element analysis is performed to obtain total stresses, σ_{SAA} and σ_{CAA}, in the shells and core of layer A due to self weight using elastic moduli with respect to total stresses, E and ν. The total stresses obtained are assumed constant between time t_o and t_A, which are the times of starting and completing the placement of layer A. After the total stresses are determined, the finite element program for pore pressure analysis is used to determine the pore pressure in every element in the core at various times. In order to evaluate deformations,

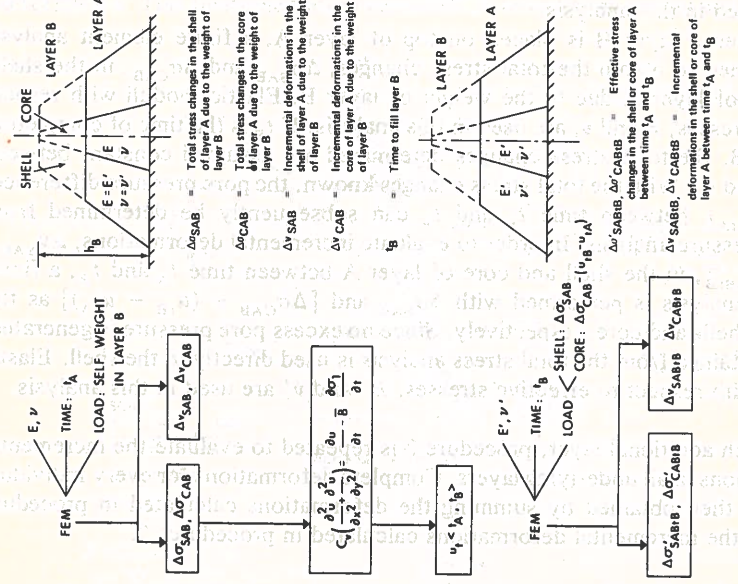


FIG. 4.—Analysis Procedures for Every Additional Layer

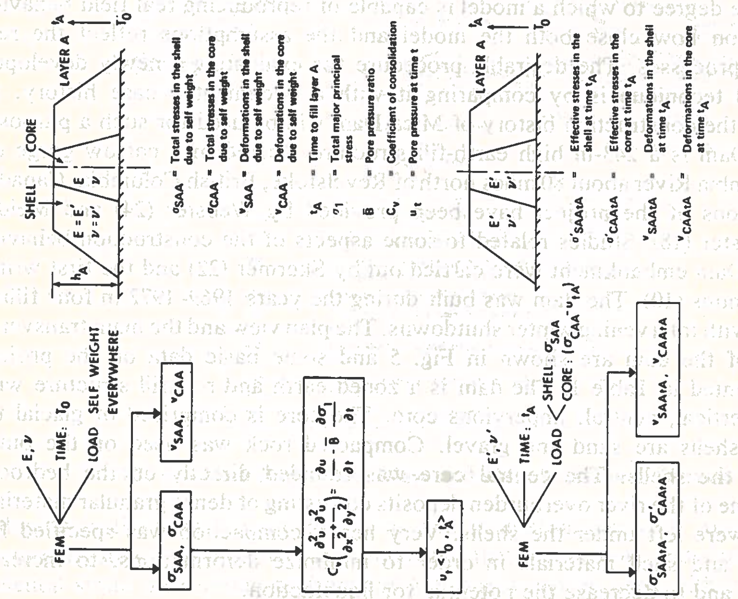


FIG. 3.—Analysis Procedure for Newly Placed Layer

v_{SAAtA} and v_{CAAtA}, in the shells and core of layer A at time t_A, a finite element analysis is performed with σ_{SAA} and $(\sigma_{CAA} - u_{tA})$ as the loads in shells and core, respectively. Elastic moduli with respect to effective stresses, E' and ν', are used in this analysis.

2. When a layer B is placed on top of layer A, a finite element analysis is performed to obtain the total stress changes, $\Delta\sigma_{SAB}$ and $\Delta\sigma_{CAB}$, in the shells and core of layer A due to the weight of layer B. Elastic moduli with respect to total stresses, E and ν, are used in this analysis. If t_B is the time of completion of layer B, the total stress changes determined are assumed constant between time t_A and t_B. With the total stress changes known, the pore pressure difference, $(u_{tB} - u_{tA})$, between time t_B and t_A can subsequently be determined from a pore pressure analysis. In order to evaluate incremental deformations, Δv_{SABtB} and Δv_{CABtB}, in the shell and core of layer A between time t_A and t_B, a finite element analysis is performed with $\Delta\sigma_{SAB}$ and $[\Delta\sigma_{CAB} - (u_{tB} - u_{tA})]$ as the loads in shells and core, respectively. Since no excess pore pressure is generated, $\Delta\sigma_{SAB}$ obtained from the total stress analysis is used directly in the shell. Elastic moduli with respect to effective stresses, E' and ν' are used in this analysis.

For each additional layer, procedure 2 is repeated to evaluate the incremental deformations in all underlying layers. Complete deformations for every individual layer are then obtained by summing the deformations calculated in procedure 1 and all the incremental deformations calculated in procedures 2.

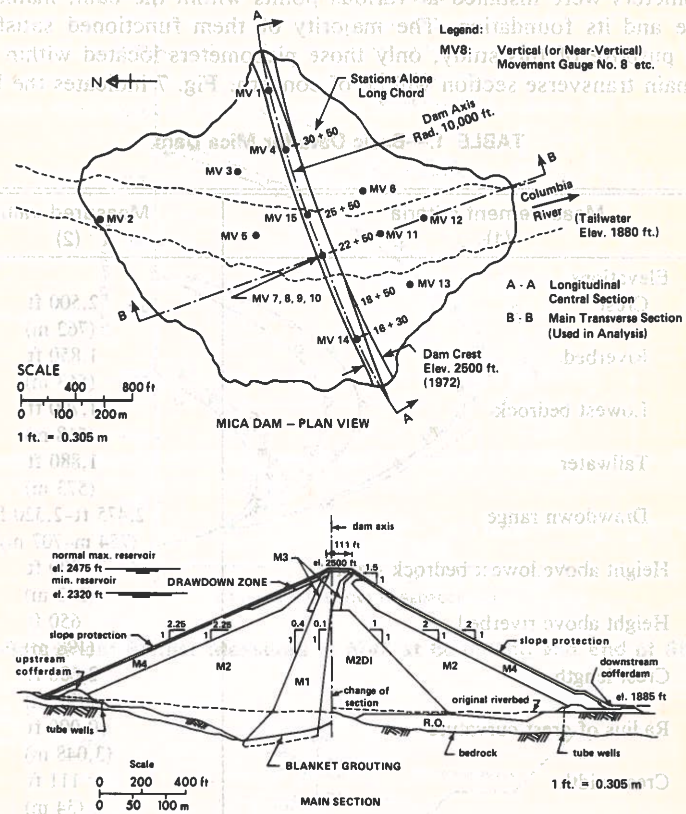
Mica Dam Case History

Every analytical model necessarily involves a number of simplifying assumptions. The degree to which a model is capable of reproducing real field behavior depends on how close both the model and the assumptions reflect the real physical processes. The desirable procedure for evaluating a newly developed analytical technique is by comparing it with a documented case history. In this case the construction history of Mica Dam will be used for such a purpose.

Mica Dam is a 243-m high earth-fill structure located in a narrow gorge on the Columbia River about 80 miles north of Revelstoke, British Columbia, Canada. Descriptions of the project have been provided by Webster (24) and Meidal and Webster (18). Studies related to some aspects of the construction behavior of Mica Dam embankment were carried out by Skermer (22) and the first writer and Simmons (10). The dam was built during the years 1969-1972 in four filling seasons with intervening winter shutdowns. The plan view and the main transverse section of the dam are shown in Fig. 5 and some basic data on the project are presented in Table 1. The dam is a zoned earth and rockfill structure with a near vertical, central, impervious core. The core is comprised of glacial till and the shells are sand and gravel. Compacted rock was used on the outer parts of the shells. The central core was founded directly on the bedrock, while some of the river overburden deposits consisting of dense granular materials and till were left under the shells. Very heavy compaction was specified for the core and shell materials in order to minimize deformations, to increase strength, and to decrease the potential for liquefaction.

A comprehensive system of instrumentation has been provided to measure horizontal and vertical movements, earth pressure, pore water pressure, and

dynamic movements in the dam (18,25). An important part of this program were measurements with vertical or near vertical movement gages (MV) shown in Fig. 5, which were assumed to provide data on vertical and horizontal deformation. This instrumentation consisted of telescoping casing and transverse



ZONE	DESCRIPTION
M1	Core, glacial till in 25 cm (10") layers
M2	Main shell, sand and gravel in 30 cm (12") layers, changed during construction to 45 cm (18") layers
M2DI	Inner zone of poorer M2 materials
M3	Core support zone, sand and gravel or rock in 15 cm (6") layers
M4	Outer shell, sand and gravel or rock in 60 cm (24") layers
Drawdown Zone	Gravel, cobbles and boulders or rock in 60 cm (24") layers
R.O.	Original River Overburden

FIG. 5.—Mica Dam—Plan View and Main Transverse Section

aluminum disks set around the casing and at intervals along it. Investigations into consistency of observed data indicated that settlements and vertical incremental strain measurements from the MV gages can be used with confidence. A typical example of settlements measured in the core is represented by the data obtained in MV 8, located in the main transverse section (Fig. 6). Despite some minor irregularities, the measured settlement profiles displayed the typical quasi-parabolic shapes. The time dependent component of settlement occurring during the winter shutdown periods is also apparent and is a significant amount. Up to 25% of the total settlement occurred under constant loads.

Piezometers were installed at various points within the dam, mainly within the core and its foundation. The majority of them functioned satisfactorily. For the purpose of this study, only those piezometers located within or close to the main transverse section will be of concern. Fig. 7 indicates the locations

TABLE 1.—Basic Data for Mica Dam

Measurement criteria (1)	Measured value (2)
Elevations	
Crest	2,500 ft (762 m)
Riverbed	1,850 ft (563 m)
Lowest bedrock	1,700 ft (518 m)
Tailwater	1,880 ft (573 m)
Drawdown range	2,475 ft-2,320 ft (754 m-707 m)
Height above lowest bedrock surface	800 ft (243 m)
Height above riverbed	650 ft (198 m)
Crest length	2,600 ft (792 m)
Radius of crest curvature	10,000 ft (3,048 m)
Crest width	111 ft (34 m)
Heel to toe length	3,100 ft (946 m)
Total volume of fill	43×10^6 cu yd (33×10^6 m^3)
Total volume of core	4.3×10^6 cu yd (3.3×10^6 m^3)

of these piezometers and also shows the variations of piezometric and fill elevations with time. From this figure, note that the measured pore pressures followed the sequence of construction seasons and stoppages reasonably well. The only exception is perhaps the first winter (1969-1970), when the pore pressures remained more or less constant or even increased slightly. There are several factors contributing to this behavior. Piezometers PP20 and PP21 were probably influenced by seepage of river water into the lower part of the core, which eventually maintained equilibrium of water levels in the core and the adjacent shells. For PE 24, the reason might be that the time of installation of these

piezometers was close to the end of the construction season. Before the equilibrium pore pressure could be established between the surrounding soil and the measuring systems, the construction season had stopped. Consequently,

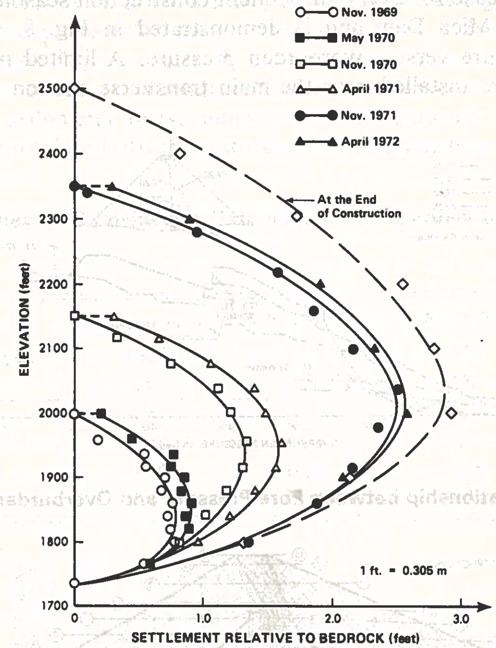


FIG. 6.—Settlement Profiles Measured in MV8 at Beginning and End of Shutdown Seasons

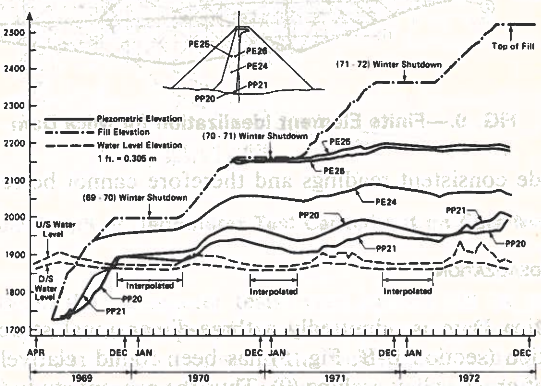


FIG. 7.—Relationships between Fill Elevation, Piezometric Elevation, and Time

the increase in observed piezometric elevation during the first shutdown season appears to be due to the process of establishing an equilibrium condition. Apart from this, the pore pressures measured after the first construction shutdown can be regarded as representative for the actual conditions in the fill.

The influence of pore pressure dissipation during the shutdown season on subsequent pore pressure development was explained by Bishop (3). The $\bar{B}$ value should decrease for every subsequent construction season. This tendency was observed at Mica Dam and is demonstrated in Fig. 8, which is a plot of the pore pressure versus overburden pressure. A limited number of earth pressure cells were installed near the main transverse section. However, they

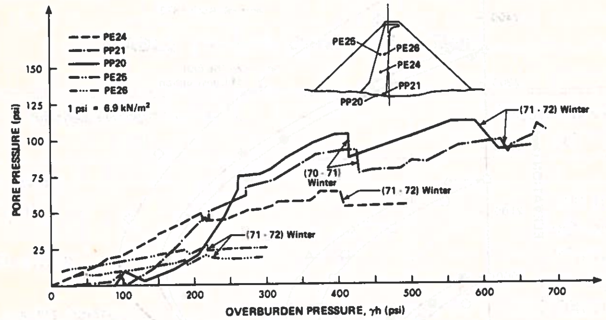


FIG. 8.—Relationship between Pore Pressure and Overburden Pressure

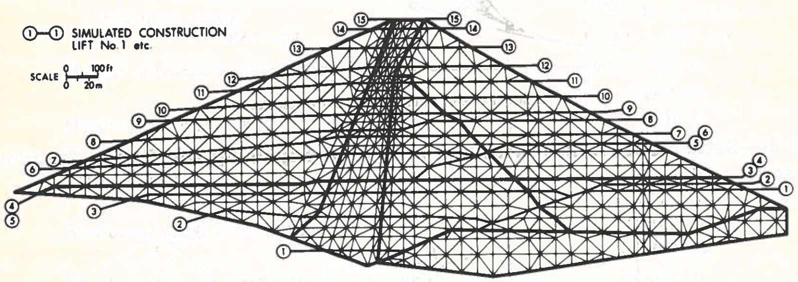


FIG. 9.—Finite Element Idealization for Mica Dam

failed to provide consistent readings and therefore cannot be relied on for this study (22).

Finite Element Idealization

While the Mica Dam is admittedly a three-dimensional structure, the main transverse section (section B-B, Fig. 5) has been found relatively uninfluenced by the effects of cross-valley arching (9). Thus the present study focuses entirely on a two-dimensional plane strain analysis of this section, which happens to be also the main instrumentation section of the dam. The finite element mesh is presented in Fig. 9. Seven different materials were distinguished within the embankment profile, on the basis of different stress-strain properties and bulk densities. The actual construction was simulated by 15 lifts. Particular attention has been paid to stress-strain representation of the soils. Settlements calculated

using nonlinear triaxial test results overestimated the measured settlements by a factor of some 2.5 to 3.5. It is obvious that deformations in an earth dam during construction do not occur under conventional triaxial stress path conditions. The stress path followed is most likely a constant or nearly constant principal stress ratio path, at least for most of the volume of a dam and certainly for a centrally located clay core laterally confined by stiff shells.

The nonlinear stress-strain relationships for the clay core were obtained from high pressure oedometer tests, supplemented by isotropic compression tests. Since both the total and effective stress-strain relationships are needed for the

TABLE 2.—Values of $\bar{B}$ Coefficient Determined from Oedometer Tests with Partial Dissipation

Construction year (1)	Values of $\bar{B}$ for fill placed during: 1969 (2)	1970 (3)	1971 (4)	1972 (5)
1969	0.52	—	—	—
1970	0.37	0.35	—	—
1971	0.25	0.23	0.35	—
1972	0.16	0.15	0.23	0.35

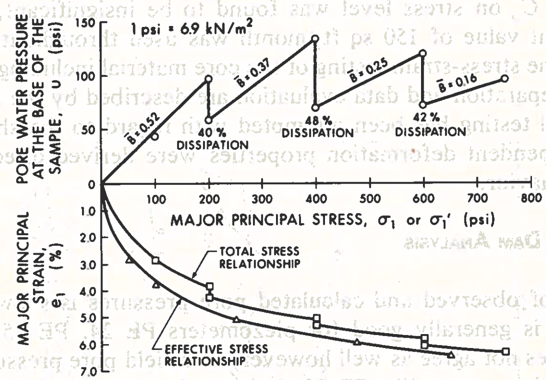


FIG. 10.—Typical Result of Oedometer Test Carried out on Core Material from Mica Dam

complete analysis, the oedometer tests were carried out as indicated in Fig. 10. The core specimens, prepared by kneading compaction to densities and moisture contents identical to those in the field, were loaded in an undrained condition with pore pressure measurements up to stress levels estimated to correspond to the end of each construction season. Since the exact stress path in the core could not be known before the analysis was actually carried out, the stress path of the oedometer test had to be based on judgment only. At each stress level simulating a completed construction season, the pore pressure in the sample was allowed to dissipate to a degree approximating the dissipation in the core during winter shutdown. The estimate of the degree of dissipation was done on the basis of field observations of pore pressures in the dam during winter seasons. Such an approach was possible because the analysis was made "post mortem." Should this method of analysis be applied at a design stage, the estimate would be more difficult. A separate preliminary pore pressure analysis might be required for this purpose. Thus, in addition to the total and effective stress-strain curves the oedometer test provides also the stress-dependent $\bar{B}$ values.

The coefficient $\bar{B}$ obviously has an important role in this type of analysis. A typical set of its values measured in an oedometer test with partial dissipation is shown in Fig. 10 for the type of core fill placed during the first (1969) construction season. This figure also indicates percentages of pore pressure dissipation observed during periods of open drainage between loading stages. Table 2 summarizes the values of $\bar{B}$ coefficient as used in the analysis. They reflect the stress dependency of $\bar{B}$ as well as the slight difference between the fill material placed in 1969 and that placed in the following years. A single constant value of $\bar{B}$ was assigned in the consolidation analysis to each volume of core fill placed during one season. This value was then changed for every subsequent season.

Since the coefficient of consolidation, C_v, is known to be stress path dependent, its value has been determined from the same oedometer tests. Values of C_v were calculated from the theoretical relationship between the percentage of pore pressure dissipation at the base of the sample and the time factor. The dependency of C_v on stress level was found to be insignificant, and thus an average constant value of 150 sq ft/month was used throughout the analysis. The details of the stress-strain testing of the core material including a description of specimen preparation and data evaluation are described by the second writer (17). No special testing has been attempted with regard to the shell materials. Their stress-dependent deformation properties were derived directly from observed field behavior.

Results of Mica Dam Analysis

Comparison of observed and calculated pore pressures is shown in Fig. 11. The agreement is generally good for piezometers PE 24, PE 25, and PE 26. The analysis does not agree as well however, with field pore pressures observed in PP21. This piezometer, like PP 20, is located about 200 ft from the plane of analysis. The calculated values for PP 21 given in Fig. 11 are those projected onto the main transverse section. It is likely that part of the discrepancy in results can be attributed to this geometrical inconsistency. However, the main factor for the higher observed values in PP 20 and PP 21 was considered to be the effect of wetting of lower parts of the core affected by upstream and downstream water levels during construction.

The results of settlements calculated from the total stress analysis and from the complete analysis (i.e., total and effective stress analyses) together with the observed values at the end of construction are shown for movement gages within the main transverse section in Fig. 12. From these results, note that the complete analysis has improved the prediction of settlements in the core. The differences between the total stress analysis (ignoring time dependent components) and the complete analysis are about 15%-20% of settlements

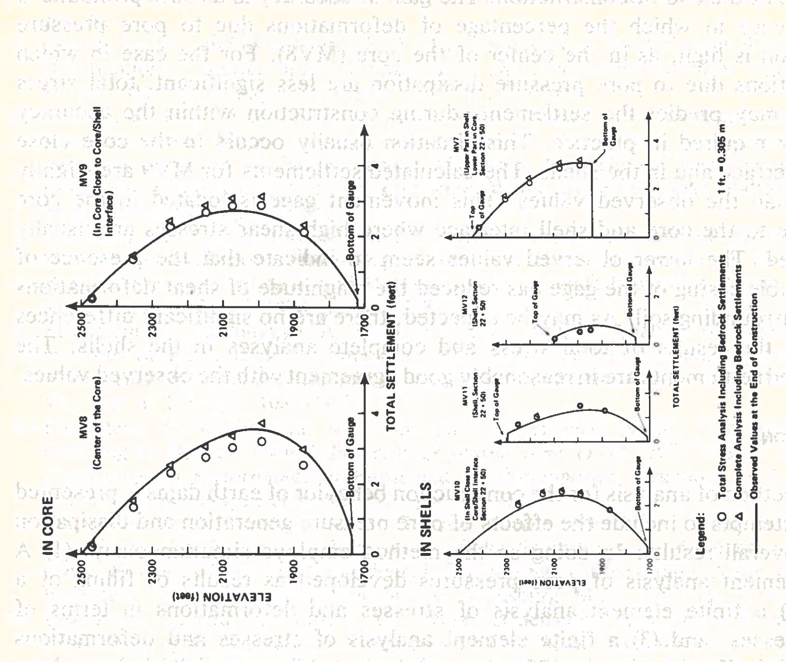


FIG. 12.—Comparison of Calculated and Observed Settlements for Movement Gages in Mica Dam

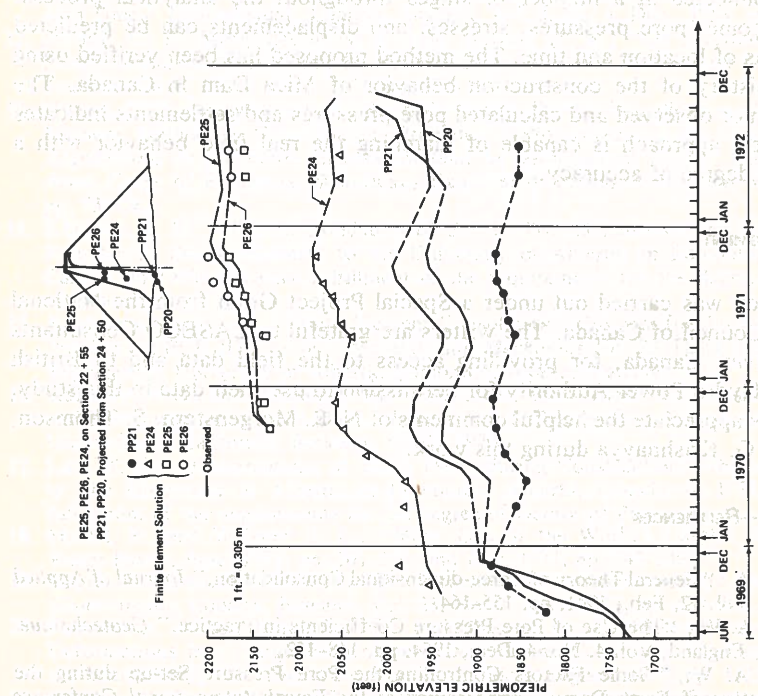


FIG. 11.—Observed and Calculated Pore Pressures for Piezometers PP 20, PP 21, PE 24, PE 25, and PE 26

observed at the end of construction. The gain in accuracy is usually pronounced for the case in which the percentage of deformations due to pore pressure dissipation is high, as in the center of the core (MV8). For the case in which deformations due to pore pressure dissipation are less significant, total stress analysis may predict the settlements during construction within the accuracy generally required in practice. This situation usually occurs in the core close to its interface and in the shells. The calculated settlements for MV9 are slightly higher than the observed values. This movement gage is located in the core but close to the core and shell interface where high shear stresses are usually developed. The lower observed values seem to indicate that the presence of less flexible casing of the gage has reduced the magnitude of shear deformations in the surrounding soil. As may be expected, there are no significant differences between the results of total stress and complete analyses in the shells. The calculated settlements are in reasonably good agreement with the observed values.

Conclusions

The method of analysis for the construction behavior of earth dams is presented which attempts to include the effects of pore pressure generation and dissipation in the overall results. In doing so the method employs simultaneously: (1) A finite element analysis of pore pressures developed as results of filling of a dam; (2) a finite element analysis of stresses and deformations in terms of total stresses; and (3) a finite element analysis of stresses and deformations in terms of effective stresses. The three analyses, while essentially independent, are interconnected at a number of stages throughout the analytical process. As an outcome, pore pressures, stresses, and displacements can be predicted as functions of location and time. The method proposed has been verified using the case history of the construction behavior of Mica Dam in Canada. The comparison of observed and calculated pore pressures and settlements indicates that the new approach is capable of matching the real field behavior with a reasonable degree of accuracy.

Acknowledgment

This study was carried out under a Special Project Grant from the National Research Council of Canada. The writers are grateful to CASECO Consultants in Vancouver, Canada, for providing access to the field data and to British Columbia Hydro Power Authority for permission to use their data in this study. The writers appreciate the helpful comments of N. R. Morgenstern, S. Thomson, and A. V. G. Krishnayya during this work.

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Appendix II.—Notation

The following symbols are used in this paper:

A = first layer of fill;
B = second layer of fill;
$\bar{B}$ = pore pressure coefficient;
B-B = main transverse section;
C_v = coefficient of consolidation;
E = Young's modulus, in terms of total stress;
E' = Young's modulus, in terms of effective stress;
e_1 = major principal strain;
FEM = finite element method;
h = height;
h_A = height of fill of layer A;
h_B = height of fill of layer B;
MV = vertical (or near vertical) movement gages;
PE = electric piezometer;
PP = pneumatic piezometer;
S = sth layer of fill;
T_o = starting time of first season fill;
T_1 = starting time of second season fill;
t = time;
t_A = time of completion of layer A;
t_B = time of completion of layer B;
t_S = starting time of first winter shutdown;
t_o = starting time;
u = pore pressure;
u_t = pore pressure at time t;
u_{tA} = pore pressure at time t_A;
u_{tB} = pore pressure at time t_B;
v_{CAA} = displacements in core in layer A due to weight of layer A;
v_{SAA} = displacements in shell in layer A due to weight of layer A;

v_{CAAtA} = displacements in core in layer A due to weight of layer A at time t_A;
v_{SAAtA} = displacements in shell in layer A due to weight of layer A at time t_A;
x, y = coordinates;
γ = bulk density;
Δu_o = initial change of pore pressure;
Δu_t = change of pore pressure at time t;
Δv_t = displacement increment at time t;
Δv_{CAB} = displacement increment in core in layer A due to adding weight of layer B;
Δv_{SAB} = displacement increment in shell in layer A due to adding weight of layer B;
Δv_{CABtB} = displacement increment in core in layer A due to adding weight of layer B at a time t_B;
Δv_{SABtB} = displacement increment in shell in layer A due to adding weight of layer B at time t_B;
$\Delta\sigma$ = change of total stress;
$\Delta\sigma_{CAB}$ = change of total stress in core in layer A due to the weight of layer B;
$\Delta\sigma'_{CABtB}$ = change of effective stress in core in layer *A* due to adding weight of layer B at time t_B;
$\Delta\sigma_{SAB}$ = change of total stress in shell in layer A due to adding the weight of layer B;
$\Delta\sigma'_{SABtB}$ = change of effective stress in shell in layer A due to adding weight of layer B at time t_B;
$\Delta\sigma'_t$ = change of effective stress at time t;
$\Delta\sigma_1$ = change of total major principal stress;
ν = Poisson's ratio, in terms of total stress;
ν' = Poisson's ratio, in terms of effective stress;
σ_1 = total major principal stress;
σ'_1 = effective major principal stress;
σ_{CAA} = total stresses in core in layer A due to weight of layer A;
σ_{SAA} = total stresses in shell in layer A due to weight of layer A;
σ'_{CAAtA} = effective stress in core in layer A due to weight of layer A at time t_A; and
σ'_{SAAtA} = effective stress in shell in layer A due to weight of layer A at time t_A.