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DISCUSSION

Proc. Paper 9002

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 x 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.

9016 STRESSES AND MOVEMENTS IN OROVILLE DAM

KEY WORDS: **Cracking (fracturing); Dams; Dams (earth);** Displacement; **Embankments; Finite element method; Instrumentation;** Loading; **Soil mechanics;** Stress strain diagrams

ABSTRACT: Oroville Dam, presently the world's highest embankment dam, was well-instrumented with several types of instrumentation to monitor its construction behavior. The results obtained from the instrumentation clearly indicated that the embankment performed very well with only small amounts of movement. However, the embankment core block did not perform satisfactorily and cracked during construction. The results of a finite element analysis, modeling the construction sequence and the nonlinear, stress-dependent material properties of the embankment soils, are presented and compared with the instrumentation results. The results of this analysis agreed well with the instrumentation results and showed that: (1) The small movements are attributable to the excellent stress-strain characteristics of the embankment soils; (2) significant load transfer occurred from the core to the adjacent coarse zones; and (3) the core block crakcing could have been anticipated if these results had been available during the early design stages.

REFERENCE: Kulhawy, Fred H., and Duncan, James M., "Stresses and Movements in Oroville Dam," *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 98, No. SM7, **Proc. Paper 9016**, July, 1972, pp. 653-665

9006 SHEAR MODULUS AND DAMPING: EQUATIONS AND CURVES

KEY WORDS: **Analysis;** Clays; **Damping; Design data; Earthquakes;** Laboratory equipment; **Repeated loading;** Sands; **Shear modulus;** Shear tests; Silts; **Soils; Stress-strain** curves; Torsion shear tests; Undisturbed samples; **Vibration**

ABSTRACT: Equations and graphs for the determination of shear modulus and damping of soils, for use in design problems involving repeated loading or vibration of soils, are presented. These equations and graphs are based on numerous laboratory tests on both remolded and undisturbed cohesive soils and on clean sands. Comparison of the measured and computed values shows good agreement. An example problem showing how these equations and curves are used is given.

REFERENCE: Hardin, Bobby O., and Drnevich, Vincent P., "Shear Modulus and Damping in Soils: Design Equations and Curves," *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 98, No. SM7, **Proc. Paper 9006**, July, 1972, pp. 667-692

9030 CONSOLIDATION OF LAYER UNDER STRIP LOAD

KEY WORDS: **Anisotropy; Consolidation;** Drainage; Finite element method; Footings; **Settlement (structural); Soil mechanics; Time**

ABSTRACT: A previously described finite element program is used to solve for the consolidation behavior of an elastic soil under strip loading. A parametric study is made of the effects of loading geometry, drainage, and material constants. The results indicate that the ratio of half width of load to depth of soil for drainage at the top (a/H) has little effect when a/H is greater than 1 and that there is some acceleration of settlement when a/H is smaller. Poisson's ratio of the soil has very little effect. The anisotropic permeability has the largest effect, and its influence is described in two charts. With these provisions, one-dimensional theory can be used in many cases.

REFERENCE: Christian, John T., Boehmer, Jan Willem, and Martin, Philippe P., "Consolidation of a Layer Under a Strip Load," *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 98, No. SM7, **Proc. Paper 9030**, July, 1972, pp. 693-707

ENGLISH-SI CONVERSION FACTORS

In accordance with the October 1970 action of the ASCE Board of Direction, which stated that all publications of the Society should list all measurements in both customary (English) and SI (International System) units, the list below contains conversion factors to enable readers to compute the SI unit values of measurements. A complete guide to the SI system and its use is available from the American Society for Testing & Materials, 1916 Race Street, Philadelphia, Pa., 19103 at a price of $1.25 per copy (minimum single order $3.00). A condensed guide to SI for civil engineering is available from ASCE headquarters.

All authors of Journal papers are being asked to prepare their papers in this dual-unit format. Until this practice affects the majority of papers published, we will continue to print this table of conversion factors:

To convert	To	Multiply by
inches (in.)	millimeters (mm)	25.40
inches (in.)	centimeters (cm)	2.540
inches (in.)	meters (m)	0.0254
feet (ft)	meters (m)	0.305
miles (miles)	kilometers (km)	1.61
yards (yd)	meters (m)	0.91
square inches (sq in.)	square centimeters (cm^2)	6.45
square feet (sq ft)	square meters (m^2)	0.093
square yards (sq yd)	square meters (m^2)	0.836
acres (acre)	square meters (m^2)	4047.
square miles (sq miles)	square kilometers (km^2)	2.59
cubic inches (cu in.)	cubic centimeters (cm^3)	16.4
cubic feet (cu ft)	cubic meters (m^3)	0.028
cubic yards (cu yd)	cubic meters (m^3)	0.765
pounds (lb)	kilograms (kg)	0.453
tons (ton)	kilograms (kg)	907.2
one pound force (lbf)	newtons (N)	4.45
one kilogram force (kgf)	newtons (N)	9.81
pounds per square foot (psf)	newtons per square meter (N/m^2)	47.9
pounds per square inch (psi)	kilonewtons per square meter (kN/m^2)	6.9
gallons (gal)	liter (dm^3)	3.8
acre-feet (acre-ft)	cubic meters (m^3)	1233.
gallons per minute (gpm)	cubic meters/minute (m^3/min)	0.004
newtons per square meter (N/m^2)	pascals (Pa)	1.00

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

STRESSES AND MOVEMENTS IN OROVILLE DAM

By Fred H. Kulhawy[1] and James M. Duncan,[2] Associate Members, ASCE

INTRODUCTION

Oroville Dam, located on the Feather River in Northern California, is presently (1972) the world's highest embankment dam. Many new developments and unique features were incorporated in this project (Gianelli, 1969) and, because of these accomplishments, the project was awarded the Civil Engineering Achievement of the Year Award for 1968 by ASCE.

The 80,000,000-cu-yd Oroville embankment, with a crest length of 5,600 ft, base width of 3,500 ft and maximum height of 770 ft, is founded on a very sound, hard amphibolite. The major zones in the embankment, shown in Fig. 1, are the inclined impervious core, the transition, the shell and the 128-ft high concrete core block. A small zone of soft clay was placed along the upstream face of the core block. Several other small zones also were placed in the embankment, but they have not been considered in this study because they appeared to have very little effect on the behavior of the embankment.

Construction of the embankment continued over a period of 5 yr. The 400-ft high cofferdam, upstream from the coreblock in Fig. 1, was constructed during 1963 and 1964; the main embankment was begun in 1965 and was completed in October, 1967.

The embankment was instrumented extensively, and the availability of these data provided an opportunity to observe whether recently developed finite element procedures could be used effectively to analyze the stresses and movements of the dam during construction. The results of this finite element

Note.—Discussion open until December 1, 1972. To extend the closing date one month, a written request must be filed with the Executive Director, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM7, July, 1972. Manuscript was submitted for review for possible publication on January 17, 1971.

[1] Asst. Prof. of Civ. Engrg., Syracuse Univ., Syracuse, N.Y.
[2] Assoc. Prof. of Civ. Engrg., Univ. of California, Berkeley, Calif.

analysis, compared to the results obtained from the instrumentation, are presented in the following sections.

FINITE ELEMENT IDEALIZATION

The analysis described herein was conducted using finite element procedures for analysis of embankments recently developed by Kulhawy, et al. (1969). This analysis employs nonlinear, stress-dependent, stress-strain relationships for the tangent modulus and the tangent Poisson's ratio of the embankment soils. The tangent modulus variation is expressed by the hyperbolic relationship proposed by Duncan and Chang (1970). The tangent Poisson's ratio variation is developed in a similar manner and is based upon a hyperbolic equation:

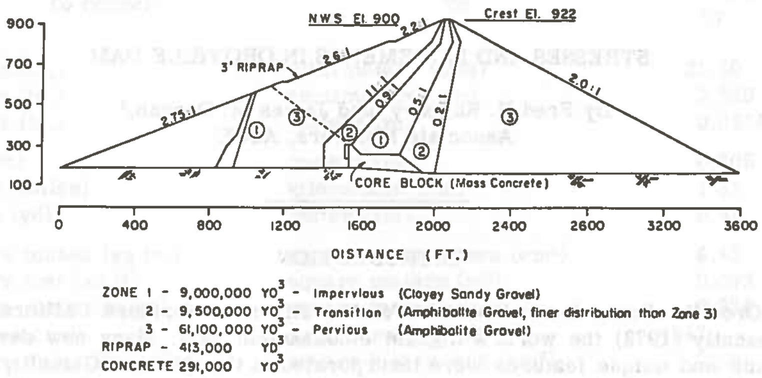


FIG. 1.—OROVILLE DAM MAXIMUM SECTION

$$\epsilon_a = \frac{\epsilon_r}{f + d\epsilon_r} \quad \text{. . . . . (1)}$$

or
$$\frac{\epsilon_r}{\epsilon_a} = f + d\epsilon_r \quad \text{. . . . . (2)}$$

in which ϵ_a = the axial strain; ϵ_r = the radial strain; f = the value of tangent Poisson's ratio at zero strain or the initial tangent Poisson's ratio, ν_i; and d = the parameter expressing the rate of change of ν_i with strain. By analyzing the published stress-strain data for numerous soils, it was found that the value of ν_i generally decreased with increasing confining pressure, σ_3, in the form:

$$\nu_i = G - F \log\left(\frac{\sigma_3}{p_a}\right) \quad \text{. . . . . (3)}$$

in which G = the value of ν_i at one atmosphere; F = the rate of change of ν_i with σ_3; and p_a = atmospheric pressure in the same units as σ_3.

Utilizing the preceding equations and the basic definition of the tangent Poisson's ratio:

$$\nu_t = \frac{\partial\,\epsilon_r}{\partial\,\epsilon_a} \quad \text{. . . . . (4)}$$

it can be shown that the resulting expression for the tangent Poisson's ratio is

$$\nu_t = \frac{G - F\log\left(\frac{\sigma_3}{p_a}\right)}{(1 - d\epsilon_a)^2} \quad \text{. . . . . (5)}$$

in which
$$\epsilon_a = \frac{(\sigma_1 - \sigma_3)}{Kp_a\left(\frac{\sigma_3}{p_a}\right)^n \left[1 - \frac{R_f(\sigma_1 - \sigma_3)(1 - \sin\phi)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]} \quad \text{. . . . (6)}$$

as given by Duncan and Chang (1970). The tangent Poisson's ratio and modulus equations were then employed in conducting incremental analyses of Oroville Dam, using the parameters shown in Table 1.

TABLE 1.—VALUES OF STRESS-STRAIN PARAMETERS FOR ANALYSIS OF OROVILLE DAM

Parameter (1)	Values Employed in Analyses: Shell[a] (2)	Transition[b] (3)	Core[c] (4)	Soft clay[b,d] (5)	Concrete[e] (6)
Unit weight, γ, in pounds per cubic foot	150	150	150	125	162
Cohesion, c, in tons per square foot	0	0	1.32[f]	0.3	216[g]
Friction angle, ϕ, in degrees	43.5	43.5	25.1[f]	13.0	0
Modulus number, K	3760	3350	345	150	137,500
Modulus exponent, n	0.19	0.19	0.78	1.0	0
Failure ratio, R_f	0.76	0.76	0.66	0.9	1.0
Poisson's ratio parameters					
G	0.43	0.43	0.30	0.49	0.15
F	0.19	0.19	-0.05	0	0
d	14.8	14.8	3.63	0	0

[a] From Marachi, et al. (1969)—Drained triaxial tests.
[b] Estimated-based on studies by Kulhawy, Duncan and Seed (1969).
[c] From Department of Water Resources (1969)—Unconsolidated-undrained triaxial tests.
[d] Zone of soft clay at upstream end of core block.
[e] After 300 days of creep—from Polivka, Pirtz and Adams (1963) and Tuthill, Adams and Mitchell (1963).
[f] c and ϕ for $(\sigma_1 + \sigma_3) < 50$ tons/ft²; $c = 10.2$ tons/ft², $\phi = 4°$ for $(\sigma_1 + \sigma_3) > 50$ tons/ft².
[g] Tensile strength of concrete ≃ 14 tons/ft² (200 psi).

The actual analyses were conducted by simulating the successive placement of layers of fill in the embankment, changing the tangent modulus and Poisson's ratio values at each placement in accordance with the stress state in each layer at a given stage of construction. The incremental analyses were then conducted in three stages, simulating the field construction sequence as closely as possible. The first finite element mesh contained 89 elements and was used to simulate construction of the concrete core block in nine lifts. The second mesh contained 111 elements and was used to simulate subsequent construction of the 400-ft high cofferdam upstream from the core block in nine lifts. The third mesh contained 249 elements and was used to simulate construction of the main embankment over the core block and against the downstream slope of the cofferdam in 12 lifts. The results obtained from the previous stages were used as input data for the subsequent stage. The foundation was considered to be rigid because the instrumentation results showed that the foundation movements were negligible compared to the embankment movements.

MOVEMENTS AND STRESSES IN EMBANKMENT

To determine the degree to which the results of the analysis reflect the behavior of the dam, the calculated movements and stresses have been com-

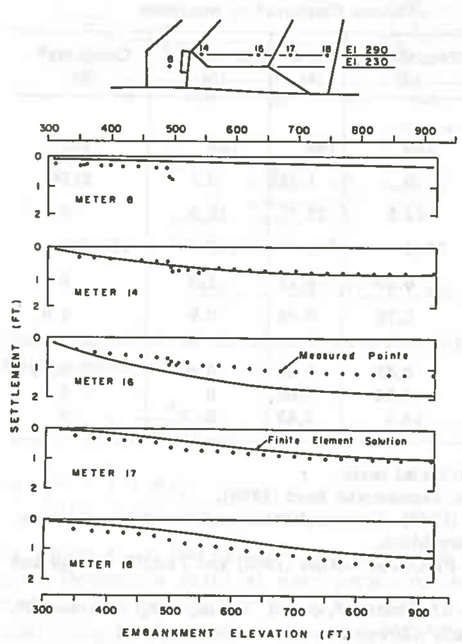


FIG. 2.—SETTLEMENTS AT EL. 250 AND EL. 290 IN OROVILLE DAM

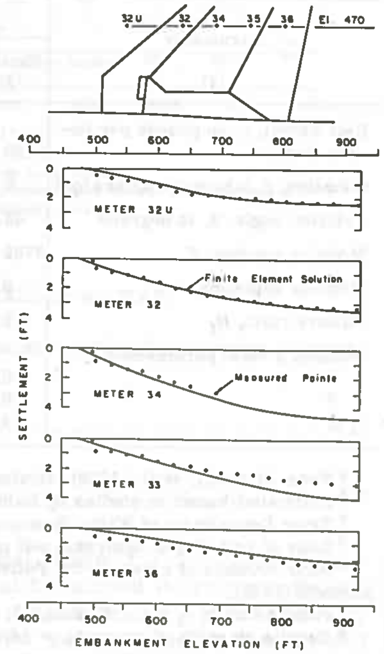


FIG. 3.—SETTLEMENTS AT EL. 470 IN OROVILLE DAM

pared to the measured movements and stresses, determined by instrumentation installed in the dam, which have been summarized by the Department of Water Resources (1967). Space limitations preclude a complete comparison, so only typical results are presented in the following sections.

Settlements.—The settlements of Oroville Dam were measured by means

of surface markers and instruments installed within the embankment during construction. Two cross-arm devices were installed in the downstream shell and 35 fluid level settlement devices were installed at 11 elevations in the

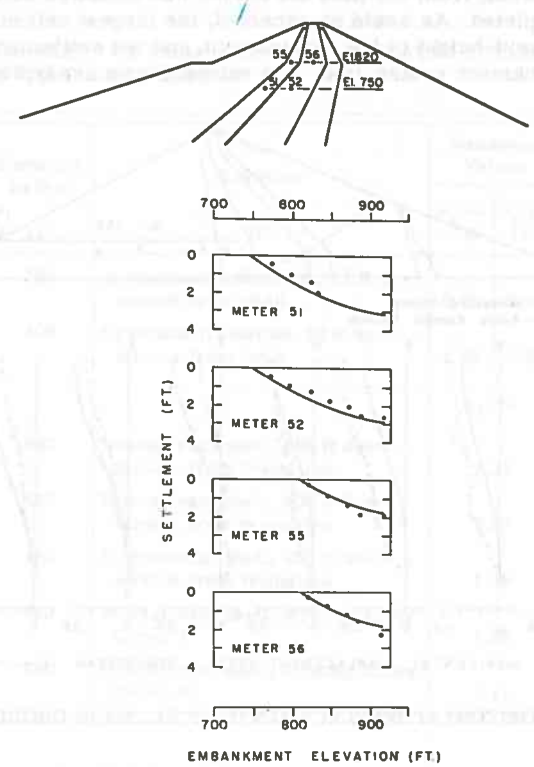


FIG. 4.—SETTLEMENTS AT EL. 750 AND EL. 820 IN OROVILLE DAM

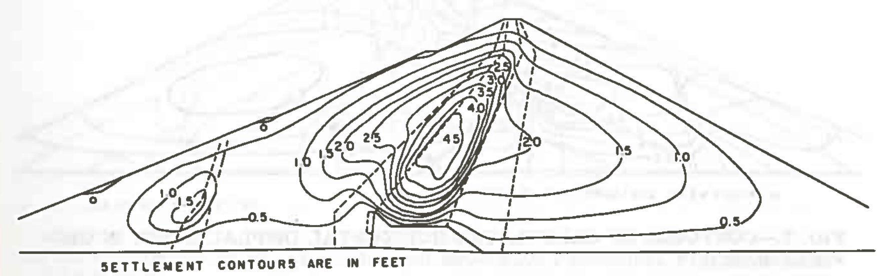


FIG. 5.—CONTOURS OF CALCULATED SETTLEMENT IN OROVILLE DAM

core, transition and upstream shell. These settlement devices, which employ sensitive measurements of fluid pressure changes as a means of measuring settlements, were used for the first time in Oroville Dam (Gordon, 1968).

Comparisons of the measured and calculated settlements at the locations of several of the fluid level settlement devices are shown in Figs. 2, 3, and 4. In these figures the settlements are plotted against the elevation of the top of the embankment, from the time the device was installed until the embankment was completed. As would be expected, the largest settlements occur at approximately mid-height of the embankment, and the settlements are largest near the embankment center line. The values shown are typical of the com-

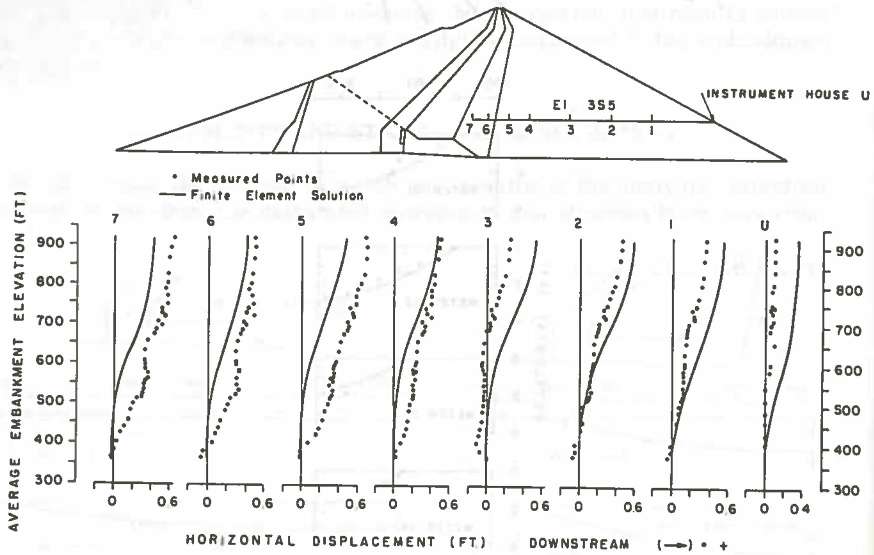


FIG. 6.—HORIZONTAL DISPLACEMENTS AT EL. 355 IN OROVILLE DAM

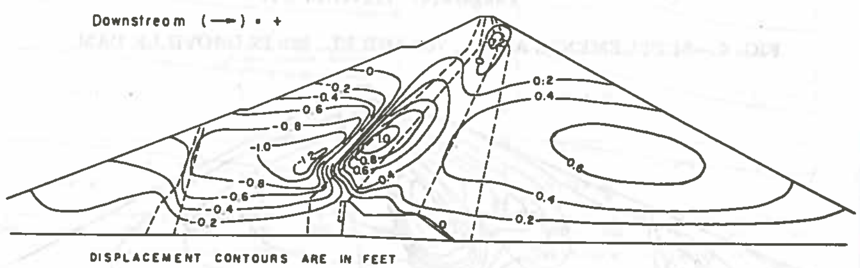


FIG. 7.—CONTOURS OF CALCULATED HORIZONTAL DISPLACEMENT IN OROVILLE DAM

parisons for all the settlement devices. On the whole, the agreement is quite good. In fact, the computed settlements were within 25 % of the observed settlements for 30 of the 35 settlement devices and for both cross-arm devices. The close correspondence in most cases, and the similarity of form between the measured and calculated variations in all cases, indicate that these pro-

cedures may be used with some confidence to calculate settlements in embankments during construction.

Contours of the calculated values of settlement in Oroville Dam are shown in Fig. 5. These values exceed 4.5 ft near the center of the core of the main

TABLE 2.—COMPARISON OF MEASURED AND CALCULATED STRESSES AT STRESS METER LOCATIONS IN OROVILLE DAM

Stress meter group	Elevation, in feet	Location	Measured Values		Calculated Values	
			$\sigma_1/\gamma h$	$\sigma_3/\gamma h$	$\sigma_1/\gamma h$	$\sigma_3/\gamma h$
(1)	(2)	(3)	(4)	(5)	(6)	(7)
A	280	Downstream transition, 20 ft upstream from shell	0.46	0.16	0.91	0.36
D	400	Upstream transition, 30 ft upstream from core	1.05	0.09	1.18	0.68
			$\sigma_y/\gamma h$		$\sigma_y/\gamma h$	
V	460	Downstream shell, 150 ft downstream from transition	1.01		0.93	
W	460	Downstream shell, 300 ft downstream from transition	1.27		1.00	
X	460	Downstream shell, 450 ft downstream from transition	1.18		1.01	
Y	560	Downstream shell, directly above Group V	1.20		0.95	
Z	580	Downstream shell, directly above Group W	1.11		1.03	

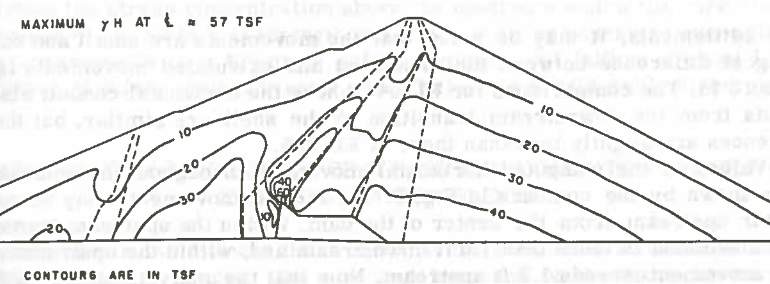


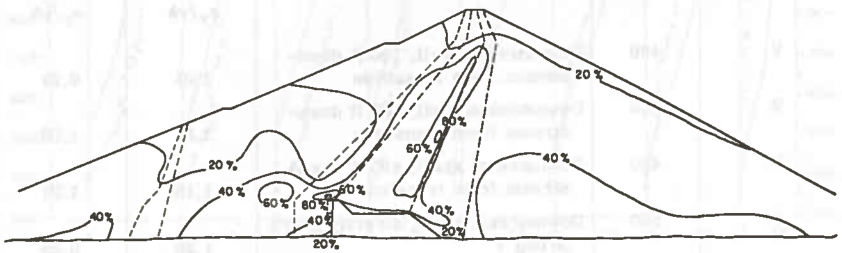
FIG. 8.—CONTOURS OF CALCULATED MAXIMUM PRINCIPAL STRESS IN OROVILLE DAM

embankment and decrease rapidly within the transition zones and the shell. There is another zone of relatively large settlement near the center of the core of the upstream cofferdam. Although the concrete core block was not assumed to be rigid, it deformed so little that for practical purposes it might be considered to be rigid in comparison with the embankment material.

From these settlement observations, it can be seen that the main core and the cofferdam core both settle with respect to the adjacent coarse zones. Furthermore, upstream from and above the core block, the shell settles with respect to the transition zone, due to the presence of the core block.

Horizontal Movements.—Horizontal movements of the dam during construction were measured by means of surface markers and special instruments installed in the embankment during construction. Two lines of horizontal movement devices were installed in the downstream shell at El. 355 and El. 540. These devices consist of horizontal conduits containing a number of steel aircraft cables, each of which is attached to an anchor embedded within the embankment. Horizontal displacements are determined by measuring movements of the cables in the instrument houses at the downstream slope (Gordon, 1968).

Fig. 6 shows the variations of the computed and observed horizontal movements at El. 355 with increasing height of the embankment. Although the observed and calculated values differ by larger percentages than in the case of



PERCENTAGE OF STRENGTH MOBILIZED = $\frac{(\sigma_1 - \sigma_3)_{MOBILIZED}}{(\sigma_1 - \sigma_3)_{FAILURE}} \times 100\,\%$

Calculated not measured

FIG. 9.—CONTOURS OF PERCENTAGE OF STRENGTH MOBILIZED IN OROVILLE DAM

the settlements, it may be noted that the movements are small and that the largest difference between the measured and calculated movements is only about 3 in. The comparisons for El. 540 where the horizontal conduit also extends from the downstream transition to the shell are similar, but the differences are slightly less than those at El. 355.

Values of the computed horizontal movement throughout the embankment are shown by the contours in Fig. 7. The largest movements may be seen to occur upstream from the center of the dam. Within the upstream transition, the movement is more than 1.0 ft downstream and, within the upstream shell, the movement exceeds 1.2 ft upstream. Note that the analysis showed a downstream movement in the upper portion of the upstream face. This direction of movement hardly could have been anticipated on an intuitive basis, but measured values of surface monuments showed that this calculated direction of movement is correct.

Stresses.—Embankment stresses were measured by means of 42 stress meters placed in the embankment during construction. These meters were installed in rosette groups at several locations to determine either the principal or vertical stresses. Study of the measured stress values, shown in

Table 2, revealed certain inconsistencies in the readings. The meters in group A indicate values of major principal stress smaller than the overburden pressure, which is incompatible with the type of load transfer indicated by the settlement observations. The meters in group D indicated a stress ratio of 11.7, corresponding to a mobilized friction angle of 57.5°, a value considerably greater than the actual angle of internal friction of the transition (43.5°). The stress meters in groups V through Z indicated values of vertical stress exceeding the overburden pressure by an average 25 %; if the vertical stress exceeded the overburden pressure by 25 % throughout the shell, considerations of vertical equilibrium show that the entire weight of the core and the transition zones would have to be supported by the shell, which is not reasonable. The stresses calculated from the finite element analysis, on the other hand, are compatible with the observed load transfer and with the strength characteristics of the embankment soils. Therefore it is the opinion of the writers that the calculations provide more reasonable values of the embankment stresses.

The distribution of the stresses throughout the dam is shown by the contours of calculated values of maximum principal stress (σ_1) shown in Fig. 8. The values of σ_1 within the core are only about two-thirds as large as those in the downstream transition and shell at the same elevation, indicating an appreciable degree of load transfer from the relatively softer core to the stiffer transition and shell. In addition, there is a zone of high values of σ_1 in the transition zone above the core block parapet and a zone of low values of σ_1 in the soft zone upstream from the core block, from which it may be concluded that the stiff core block and the adjacent soft zone had a large influence on the stresses in the neighboring portions of the dam. The calculated values of minimum principal stress and maximum shear stress were found to vary in a similar manner.

The severity of the stress conditions may be evaluated by contours of the type shown in Fig. 9, which indicate computed percentages of strength mobilized throughout the dam. The zones in which the greatest portions of the shear strength are mobilized are at the downstream edge of the sloping core, and within the stress concentration above the upstream end of the core block. The values of percentage of strength mobilized in these zones are about 80 %, which corresponds to a factor of safety against local failure of about 1.25. The factor of safety against overall shear failure is considerably higher.

COMPARISON OF OBSERVED CRACKING AND COMPUTED TENSILE STRESSES IN CORE BLOCK

During construction it was discovered that the core block contained a number of longitudinal cracks. The size and locations of these cracks varied somewhat from monolith to monolith, many being barely discernable and the widest being about 4 in. All of the cracks were located in the zone extending from the reentrant corner to the upstream edge of the core block, and most had an approximately vertical orientation over most of their length. These cracks were located by core drilling and were grouted later to insure the integrity of the core block.

To determine if the cracking might have been anticipated on the basis of finite element analyses of the type described herein, the results of this anal-

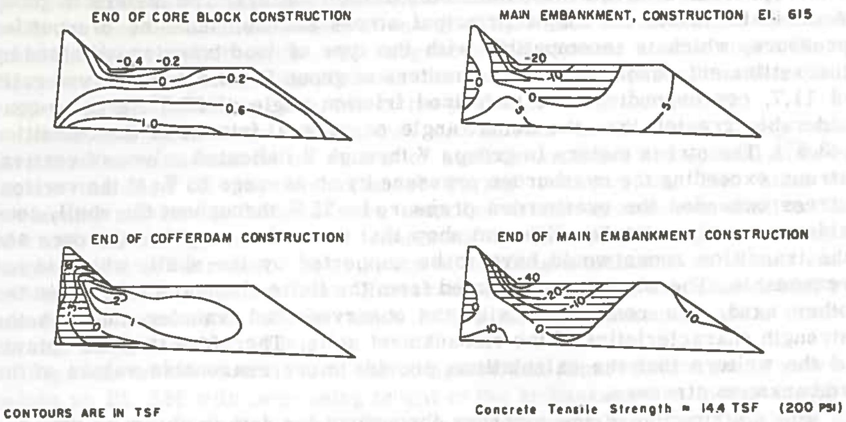


FIG. 10.—CONTOURS OF MINOR PRINCIPAL STRESS IN CORE BLOCK OF OROVILLE DAM AT FOUR CONSTRUCTION STAGES

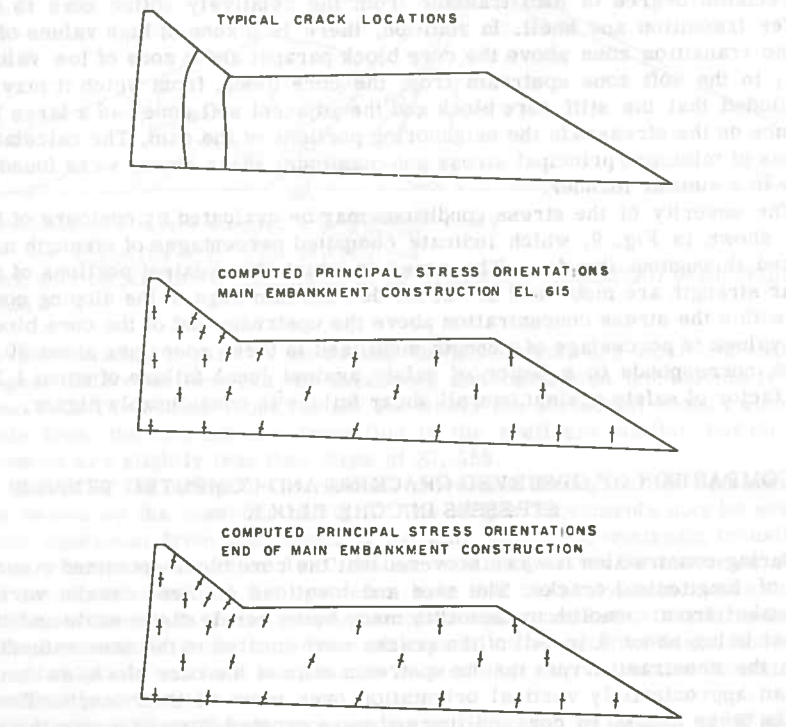


FIG. 11.—TYPICAL CRACK LOCATIONS AND COMPUTED PRINCIPAL STRESS ORIENTATIONS IN CORE BLOCK OR OROVILLE DAM

ysis were used to determine the tensile stresses in the core block at each stage during construction of the dam. Only the section shown in Fig. 1 was considered in the analysis. It was assumed that the core block concrete was linear, and crack propagations were not simulated in the analysis. Instead, development of cracks was inferred on the basis of calculated values of tensile stress exceeding the tensile strength of the concrete, and crack orientations were inferred from the calculated orientations of the planes subjected to the greatest values of tensile stress.

The magnitudes of the tensile stresses in the core block at four stages of construction are shown by the contours of minor principal stress shown in Fig. 10. These results show that, when the main embankment had reached El. 615, as shown at the upper right of Fig. 10, the largest calculated value of tensile stress was about 20 tons/ft^2, a value exceeding the tensile strength. By the end of construction the maximum calculated value had increased to more than 40 tons/ft^2, or about three times the tensile strength of the concrete. On the basis of this analysis it may be concluded that the tensile stresses around the reentrant corner of the core block became large enough to cause cracking of the concrete about the time the main embankment had reached El. 600. These tension cracks would be expected to develop on the planes of maximum tensile stress, which, as shown in Fig. 11, are in close agreement with crack orientations observed in many monoliths; the longest of the two lines from each of the crosses, representing the plane on which the minor principal stress acts, may be seen to have very nearly the same orientations as the typical cracks shown in the upper part of the figure.

CONCLUSION

The studies described herein indicate that the results of nonlinear finite element analyses, conducted using properties measured under appropriate laboratory test conditions and incremental analysis procedures, are in good agreement with the actual behavior of Oroville Dam. The calculated settlements and horizontal movements were found to be in substantial agreement with those determined by means of instruments installed in the dam during construction. The calculated values of stress in the embankment were generally not in good agreement with the values measured using soil stress meters; it was noted, however, that in some respects the measured values were not consistent with the strength characteristics of the embankment materials and the requirements of vertical equilibrium. The calculated stresses, however, are consistent with the strength characteristics and the requirements of equilibrium.

Analysis of the stresses in the concrete core block showed that zones of tension existed in the core block at several stages during construction. During the final stages of construction, as the main embankment approached completion, the calculated tensile stress increased to values which exceeded the measured tensile strength of the core block concrete. Both the magnitude and orientations of these tensile stresses are consistent with the observed cracks in the core block.

Analyses of the type employed in this study can be used very effectively in connection with instrumentation studies. Before construction these analyses provide information which would be very useful for planning instrument types and locations. During and after construction the analytical results may be checked by comparison with the observed behavior, and may be used to examine the behavior of the dam at locations where no instruments were installed.

ACKNOWLEDGMENT

This study was conducted at the University of California, Berkeley, under the sponsorship of the California Department of Water Resources, whose support is gratefully acknowledged. Ellen McKeon typed the text and Gloria Pelatowsky drafted the figures.

APPENDIX I.—REFERENCES

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APPENDIX II.—NOTATION

The following symbols are used in this paper:

c = total stress cohesion intercept;
d = rate of change of initial tangent Poisson's ratio with strain;

F = rate of change of initial tangent Poisson's ratio with confining pressure;
G = initial tangent Poisson's ratio at one atmosphere;
H = embankment height;
h = height from stress meter to embankment slope face;
K = modulus number;
n = exponent for stress-dependent modulus;
p_a = atmospheric pressure;
R_f = failure ratio;
γ = unit weight;
ϵ_a = axial strain;
ϵ_r = radial strain;
ν_i = initial tangent Poisson's ratio;
ν_t = tangent Poisson's ratio;
σ_y = vertical stress;
σ_1, σ_3 = major, minor principal stress; and
ϕ = angle of internal friction.

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DIVISION NAME CHANGE

The Technical Activities Committee, at its July 9-10, 1973 meeting, held in Tulsa, Oklahoma, approved the change in name of the Soil Mechanics and Foundations Division to the Geotechnical Engineering Division. However, we are continuing to use the "old" name for the Journals for the balance of 1973. The January 1974 issue will carry the new name.

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DISCUSSION

Proc. Paper 10038

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

TECHNICAL NOTES

Proc. Paper 10042

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.

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STRESSES AND MOVEMENTS IN OROVILLE DAM[a]

Closure by Fred H. Kulhawy[4] and James M. Duncan,[5] Associate Members, ASCE

The writers wish to thank Skermer for his discussion and agree with him that material property evaluation is one of the most important aspects of a logically conducted finite element analysis for an earth embankment. A single value of modulus or Poisson's ratio, which is representative of the behavior of a given embankment material, is extremely difficult, if not impossible, to determine. Yet if one knows, for example, the displacements at a given point or two, it is possible to determine combinations of the modulus and Poisson's ratio which will match the known displacements. However, the justification for the use of these "fitted" values, in a design analysis prior to construction, is very difficult to make simply because the material behaves in a nonlinear and stress-dependent manner and this behavior is not being modeled.

This is where the main advantage lies in the techniques described in the paper. The analysis is conducted on a step by step basis simulating the fill placement and at each step a new tangent modulus and tangent Poisson's ratio is computed from the modulus equation given by Duncan and Chang (1) and the Poisson's ratio equation given in the paper. The parameters for the equations are obtained from triaxial test data on samples prepared in accordance with field specifications. For the Oroville core material, the samples were impact compacted to field density and water content and, for the Oroville shell material, the 3-ft diam samples were vibratory compacted to field density. The writers feel that sample preparation and evaluation in this manner is the most realistic for construction analysis of embankments and that resilient moduli are not applicable to this case because the repetitive loading associated with resilient moduli is not realized in embankment construction.

[a]July, 1972, by Fred H. Kulhawy and James M. Duncan (Proc. Paper 9016).
[4]Assoc. Prof. of Civ. Engrg., Syracuse Univ., Syracuse, N.Y.
[5]Prof. of Civ. Engrg., Univ. of California, Berkeley, Calif.