CONTRACT REPORT S-71-11

CRACKING OF EARTH AND ROCKFILL DAMS

COMPARISON OF OBSERVED AND THEORETICAL TENSILE STRAINS IN THE CRESTS OF TWO EARTH AND ROCKFILL DAMS

by

S. W. Covarrubias

April 1971

Sponsored by Office, Chief of Engineers, U. S. Army

Conducted for U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

Under Contract No. DACW 39-69-C-0029

By Harvard University, Cambridge, Massachusetts

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ARMY-MRC VICKSBURG, MISS.

PREFACE

The work described in this report was performed under Contract No. DACW 39-69-C-0029, "Cracking of Earth Dams," between the U. S. Army Engineer Waterways Experiment Station (WES) and Harvard University. The contract was sponsored by the Office, Chief of Engineers, U. S. Army, under Engineering Studies Item ES-544, "Cracking of Earth Dams."

The general objective of this research, which began in 1968, was to investigate by the finite element method the factors that influence cracking in earth dams. The project was administered by the President and Fellows of Harvard University and was conducted under the supervision of Arthur Casagrande, Professor of Soil Mechanics and Foundation Engineering. Two reports have been issued previously under this contract. This report was prepared by S. W. Covarrubias.

The contract was monitored at WES by Mr. J. B. Palmerton, Rock Mechanics Section, Soil and Rock Mechanics Branch, Soils Division. Mr. J. P. Sale was Chief of the Soils Division during the preparation and publication of this report. Contracting Officers were, successively, COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE, Directors of WES. Technical Director of WES during this time was Mr. F. R. Brown.

FOREWORD

The investigations reported herein were performed in fulfillment of Contract No. DACW 39-69-C-0029 between the U. S. Army Engineer Waterways Experiment Station and Harvard University, dated 26 March 1969, and of the modification to this Contract effective 15 August 1969. This is the last report on investigations performed under this Contract, which stated the purpose and scope of this research as follows:

> "... to investigate factors influencing the development of cracks in earth dams, using the finite element method to determine stress and strain distributions in earth dams for a variety of typical boundary conditions (in particular various shapes of abutments), and of stress-strain properties of the materials in the dam and its foundations. Dams which have cracked will be analyzed to establish empirical correlations between analytical results and actual performance. "

This scope was later enlarged to include the following studies:

1) Investigation of factors affecting the development of tension zones around conduits in earth embankments.

2) Investigation of tension zones caused by a rigid cutoff wall beneath an earth dam.

3) Investigation of additional case records to compare observed tensile strains along the crest with the results of analyses by means of the finite element method.

The original Contract was fulfilled by the author's doctoral thesis entitled "Cracking of Earth and Rockfill Dams" {1}*. Items (1) and (2) of the enlarged scope of the Contract were fulfilled by the report entitled "Tension Zones in Embankments Caused by Conduits and Cutoff Walls" {2}; item (3) is fulfilled by the present report.

Work on this project was conducted by Sergio W. Covarrubias, formerly Research Fellow at Harvard University and presently Research Professor at the Institute of Engineering, National University of Mexico, under the direction of Professor Arthur Casagrande.

The information on the performance of Summersville Dam was made available by the U.S. Army Corps of Engineers and on the performance of Mattmark Dam by Professor Casagrande.

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SYNOPSIS

The purpose of the investigation is to compare the longitudinal strains observed along the crests of Summersville and Mattmark Dams with the results of analyses using the finite element method. All materials were assumed to be linearly elastic, with equal properties in tension and compression. The only load considered was the weight of the embankment.

The results of the analyses show good agreement between the measured and the computed strains. Similar results were also obtained in previous investigations by the author. It is concluded that this method of analysis is a meaningful tool in designing earth and rockfill dams.

I INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this study was to compare the actually observed strains along the crest of two dams with the strains derived by means of the finite element method in accordance with the procedure described in {1}.

The investigation reported herein included the analyses of Summersville Dam in U.S.A. and Mattmark Dam in Switzerland. For the latter dam, it was possible to make comparisons with measurements for two sets of conditions:

1) during the period when construction of the dam was halted at an elevation 20 m below the final crest elevation, and
2) after the dam was completed .

1.2 BASIC APPROACH AND METHOD OF ANALYSIS

The studies under Contract No. DACW 39-69C-0029 are based on Professor Casagrande's concept that for the purpose of investigating tension zones and cracking in earth dams it is of advantage to assume that all materials are linearly elastic because such simplification of the stress-strain properties exaggerates the magnitude of the tensile stresses without significantly changing the geometry of the tension zones and the locations of the maximum tensile stresses as compared to those that developed in actual dams. This hypothesis has been ~~thoroughly investigated~~ by the author in his doctoral thesis {1}, where he compared observational data of several dams with the results of finite element analyses of those dams and found good agreement. Similar good agreement found in the present investigation further demonstrates the usefulness of this approach.

The method of analysis and the computer program used are the same as described in Chapter 3 and Appendix of Ref. {1}. All materials are assumed to be linearly elastic, with equal properties in tension and in compression. The only load considered is the weight of the embankment, and it is assumed to be applied in a single lift.

II SUMMERSVILLE DAM

Barnes {3} described design and construction of Summersville Dam. It is a rockfill dam with a maximum height of 393 ft above foundation, a length of 2,280 ft and it is arched upstream on a 3,200-ft radius. The exterior slopes are 1 on 2.25 upstream and 1 on 1.75 downstream for the top 100 ft and 1 on 2.25 below. The dam is located on the Gauley River on the western slope of the Appalachian mountain range, in West Virginia. The entire dam is founded directly on the interbedded sandstone and shale layers that form the abutments and foundation.

Fig. 1 shows typical sections and a plan view of the dam. The core consists of two zones: an upstream zone of lean clay, and a downstream zone of clayey silts and silty sands. The core materials were compacted at a water content averaging slightly above the AASHO standard optimum.

The upstream and downstream rockfill zones consist of compacted sandstone. They are divided into three subzones insofar as layer thickness and maximum rock sizes are concerned. The subzones nearest to the core were placed in 12-in lifts with maximum rock size of 9 in. The next adjacent subzones were placed in 24-in lifts with maximum rock size of 18 in. The outside subzones were placed in 36-in lifts with a 24-in maximum size.

Construction of the dam was started in June 1961; it was completed in November 1964. In August 1965, i.e., nine months after the dam was completed, monuments for observing settlements and longitudinal strains were installed along the upstream and downstream edges of the crest of the dam and along one countour line on the upstream slope and along one countour line on the downstream slope, as shown in Fig 1. One month after the monuments were installed, filling of the reservoir was started; the water level reached the elevation of the spillway crest, which is about 20 ft below the crest of the dam, in March 1966.

The longitudinal displacements of the monuments along the crest for the period August 1965, i.e. about one month before reservoir filling was started, until June 1968, are presented in Table I. In Fig. 2 are plotted the longitudinal strains since August 1965, computed by using the average of the observations on each opposite pair of monuments for the following dates:

1. On 20 Sept 1965, when filling of the reservoir was started.
2. On 27 April 1966, about one month after the reservoir level reached the spillway crest for the first time.
3. On June 21, 1968.

It is emphasized that the strains plotted in Fig. 2 do not include the strains which developed during construction and during the first 9 months after completion of the dam. It is noteworthy that during the 35-day period between 16 Aug and 20 Sept 1965, the longitudinal strains show clearly the tension and compression zones along the crest of the dam, with slightly more than one-third of the length of the dam on each side in tension. From the strains which developed within the short period, it can be concluded that the maximum tensile strains during the 9 preceding months, since completion of the dam, have probably exceeded 0.1 %. However, no tension cracks were observed during that period. The additional tensile strains which developed during the filling of the reservoir did cause extensive longitudinal cracking, but again no transverse cracks were observed. However, the fact that the length of the tension zones decreased substantially during the filling of the reservoir while the maximum tensile strains increased, as can be seen in Fig. 2, suggests that transverse cracks may have developed. It has been shown {1, Chapter 6} that the development of cracks reduces the length of tension zones.

For the purpose of applying the finite element method to the determination of the shape of the strain distribution along the crest of the dam, the magnitude of the assumed modulus of elasticity is not important, i.e., one obtains the same geometry of the tension zones irrespective of the numerical value of the modulus. A Modulus of 1000 tsf, Poisson's ratio of 0.35 and a weight of 120 pcf were assumed in the analysis of the left half of the dam as shown in Fig. 3.

Fig. 3(a) shows the distribution of principal stresses in the dam. It can be seen that a tension zone develops along the crest of the dam over the steep portion of the abutment. The tension zone is about 530 ft long and it has a maximum depth of about 60 ft. The maximum tensile stress of about 2.1 tsf occurs at the crest approximately 560 ft from the abutment, i.e. almost at the crest monument No. 2 and roughly where the tension zone is deepest.

Fig. 3(b) shows the distribution of the longitudinal strains along the crest as determined from: (1) the finite element analysis, and (2) the field measurements which are shown in Fig. 2. It can be seen that the limit of the tension zone determined from the finite element analysis lies between the limits corresponding to the observed strains before and after filling the reservoir.

Considering that the measurements on the crest include only the effect of filling of the reservoir, and that the spacing between observation points is excessive for an accurate representation of the strain distribution along the crest, it is felt that the shapes of the theoretical and the measured strain distribution compare favorably.

III. MATTMARK DAM

Gilg and Gerber {4} have described the features of Mattmark Dam, and Gilg {5} has described its performance. It is a 120-m-high rockfill dam with a 780-m-long crest, average slopes of 1 on 1.6 downstream and 1 on 1.9 upstream and with a wide inclined core. The dam is located in the upper Rhone Valley in Switzerland. Fig. 4 shows sections of the dam. The dam is founded on glacial deposits and compressible alluvial and lucustrine sediments with a maximum depth of about 100 m.

Fig. 5 shows the grain size distribution of the materials in the dam. The core of the dam consists of non-plastic glacial till compacted in layers 30 to 40 cm thick with 4 to 5 coverages of a 40-ton rubber-tired roller and at a water content near the optimum AASHO standard. The downstream shell of the dam consists of coarse moraine material.

Construction of the dam started in 1962. In October 1965, about two months after construction of the dam had been stopped (because of an accident), and when the elevation of the core was about 20 m below the design crest elevation, transverse cracks were observed on the surface of the core. They were located at a distance of about 20 m and 30 m from the right abutment. The maximum width of the cracks at the surface was about 4.5 cm. The depth of the cracks was explored by means of shafts, and it was found to be approximately 5 m. The cracks were backfilled, instrumented with displacement gages installed across them. The dam was slowly finished to the final crest elevation in a period of two years, and filling of the reservoir proceeded cautiously in several stages. Series of 3-meter-long gages were installed parallel to the axis of the dam, at levels 17 m and 7 m below the crest along the tension zones near the abutment. Surface monuments were installed on the slopes and along the crest for observation of settlements and horizontal movements.

Fig. 6 shows the measured horizontal longitudinal strains which developed after the dam was finished, at the following elevations: (a) along the crest, (b) 7 m below the crest, and (c) 17 m below the crest. With the exception of the narrow zones where the cracks had developed near the right abutment, these and other measurements performend after the dam was completed indicate that all movements within the dam have practically stopped soon after the first complete filling of the reservoir. No open cracks have been observed on the surface after completion of the dam. It is not known whether the original cracks continue to widen at some depth below the crest.

Based on measurements of the settlements at the base and along the crest of the dam, the author estimated that the materials in the foundation were about four times more compressible than the materials in the dam. Using a Young's modulus of 400 kg/sq cm for the foundation materials and 1,600 kg/sq cm for the dam, a Poisson's ration of 0.35 for the dam and the foundation, a unit weight of 2,500 kg/cu m for the materials in the dam, and assuming an homogeneous section, a finite element analysis was performed for a portion of the longitudinal section of the dam. The analysis was carried out for two stages, the first one for the elevation where construction was interrupted and cracks observed, and the second one for the completed dam.

Fig. 7(a) shows the distribution of principal stresses for the first stage investigated. A rather large tension zone, 150 m wide by 36 m deep, occurs along the crest adjacent to the right abutment. Fig. 7(b) shows the distribution of longitudinal strains along the crest, which is quite uniform along the 100 m adjacent to the abutment, with two maxima, one near the abutment which corresponds to a stress of about 10 kg/sq cm and another of about 8.6 kg/sq cm approximately 80 m from the abutment.

Fig. 8 shows the distribution of principal stresses after the dam was completed. The tension zones extend now into the new layer, with the maximum tensile stress of about 4.5 kg/sq cm on the crest 18 m from the abutment. The tensile stresses within the first stage tension zone are now smaller but still of a substantial magnitude.

Fig. 9 shows the observed distribution of horizontal longitudinal strains (a) along the crest, (b) 7 m below the crest, and (c) 17 m below the crest. It can be seen that these distributions show maxima at the same locations where the field measurements plotted in Fig. 6 show strain concentrations.

IV. CONCLUSION

The investigations reported herein demonstrate that the zones of tensile strains and the cracking which were observed in Summersville and Mattmark Dams can be analyzed qualitatively by means of the finite element method. This conclusion is in agreement with the results of a previous investigation by the author, {1}, where he had analyzed the performance of four other earth dams.

LIST OF REFERENCES

1) Covarrubias, S. W. (1969), "Cracking of Earth and Rockfill Dams, " WES Contract Report S-69-5; also Harvard Soil Mechanics Series No. 82, Cambridge, Mass.

2) Casagrande, A. and Covarrubias, S. W. (1970), "Tension Zones in Embankments Caused by Conduits and Cutoff Walls, " WES Contract Report S-70-7; also Harvard Soil Mechanics Series No. 85, Cambridge, Mass.

3) Barnes, J. N. (1964), "Design and Construction of Summersville Dam," Transactions, 8th International Congress on Large Dams, Edinburgh, Vol. 3, pp. 711-729.

4) Gilg, B. and Gerber, F. P. (1961). "La Digue de Mattmark Essais et Etudes Preliminaires," Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, Vol. 2, pp. 607-611.

5) Gilg, B. (1970), "Apparition des Fissures dans la Digue de Mattmark," Transactions, 10th International Congress on Large Dams, Montreal, Vol. I, pp. 189-206.

TABLE I

MEASURED RELATIVE DISPLACEMENTS* BETWEEN MONUMENTS ALONG CREST OF SUMMERSVILLE DAM

UPSTREAM SIDE

DATE	A8-A7	A7-A6	A6-A5	A5-A4	A4-A3	A3-A2	A2-A1
16/08/65	0	0	0	0	0	0	0
20/09/65	.11	.08	-.09	-.05	-.02	.04	.08
06/10/65	.17	.01	-.05	-.03	-.04	.04	.16
21/10/65	.15	.0	-.02	-.05	-.10	.01	.12
03/01/66	.18	-.09	-.03	-.01	-.08	.06	.18
17/01/66	.17	-.08	-.10	-.02	-.09	-.03	.18
21/03/66	.22	-.03	-.18	-.05	-.15	-.11	.23
27/04/66	.23	-.07	-.13	-.08	-.15	-.14	.20
12/01/67	.33	-.18	-.18	0	28	-.08	.31
27/06/67	.38	-.16	-.21	-.14	-.16	-.17	.34
01/02/68	.49	-.22	-.18	-.14	-.29	-.19	.47
21/06/68	.54	-.14	-.20	-.24	-.28	-.18	.48

DOWNSTREAM SIDE

DATE	B8-B7	B7-B6	B6-B5	B5-B4	B4-B3	B3-B2	B2-B1
16/08/65	0	0	0	0	0	0	0
20/09/65	.17	.02	-.12	-.06	-.10	.11	.07
06/10/65	.13	.07	-.10	-.06	-.11	.07	.09
21/10/65	.16	.04	-.07	-.08	-.14	.07	.06
03/01/66	.15	0	-.05	-.09	-.12	-.01	.15
17/02/66	.13	0	-.16	-.04	-.17	-.01	.19
21/03/66	.18	-.01	-.25	-.04	-.20	-.09	.15
27/04/66	.19	-.13	-.21	-.08	-.24	-.09	.17
12/01/67	.35	-.09	-.26	-.07	-.28	-.14	.30
27/06/67	.39	-.13	-.26	-.12	-.25	-.19	.36
01/02/68	.49	-.14	-.36	-.11	-.28	-.22	.44
21/06/68	.53	-.16	-.20	-.17	-.43	-.19	.40

* Displacement in feet.
Filling started on 20/09/65.

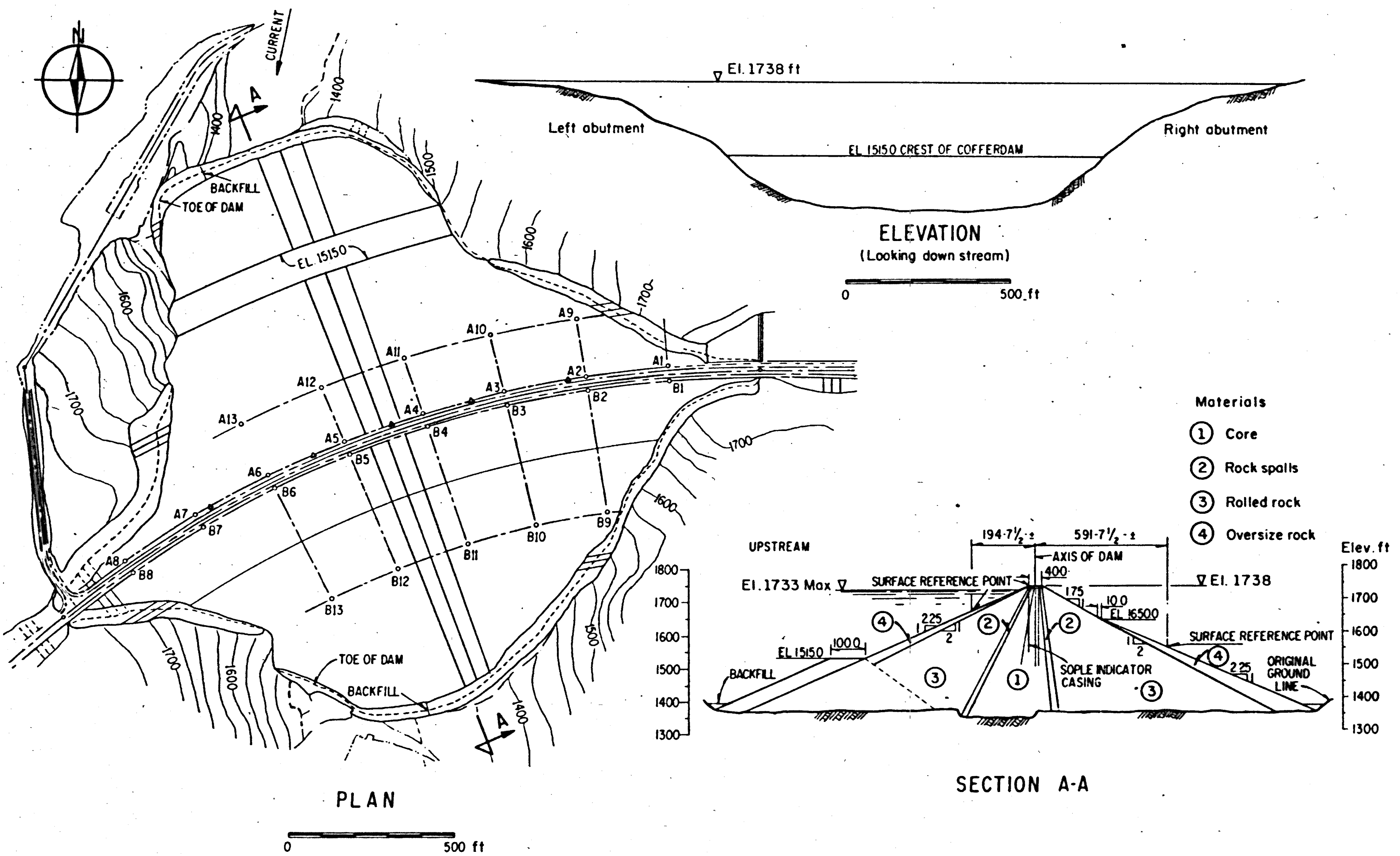


Fig 1. Plan view and cross-sections of Summersville Dam

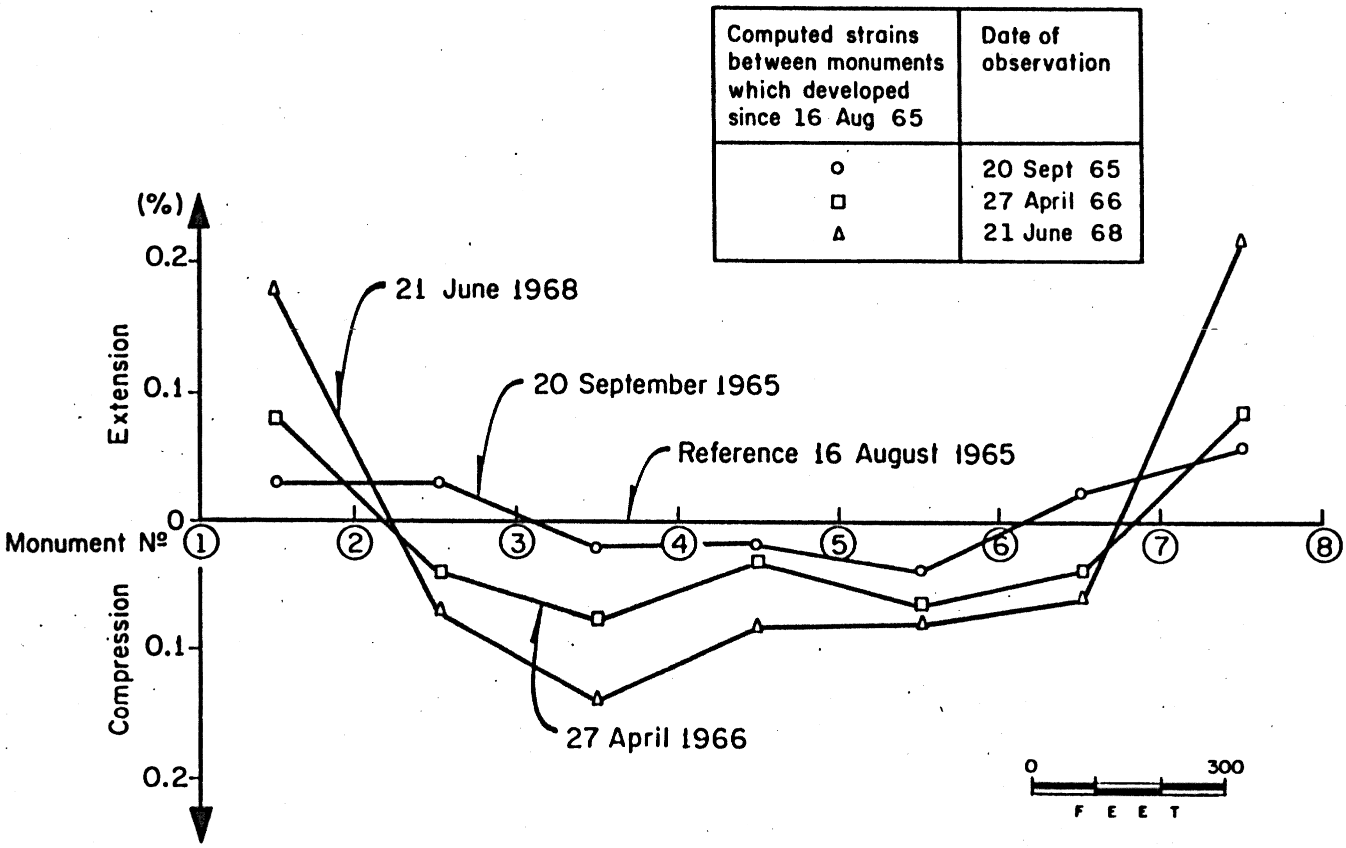


Fig 2. Longitudinal strains along crest of Summersville Dam

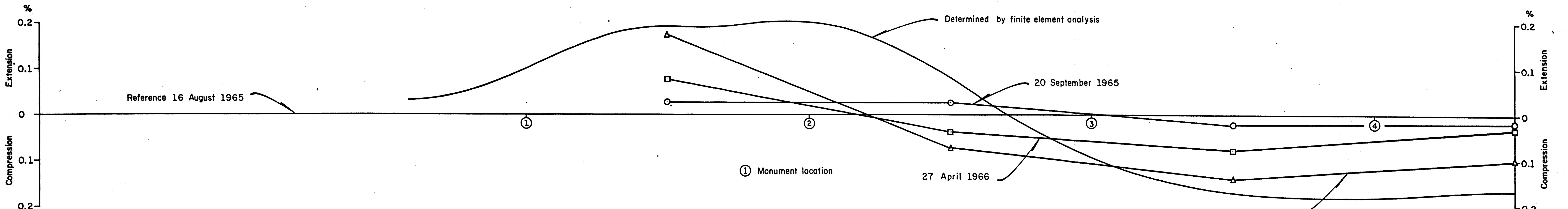
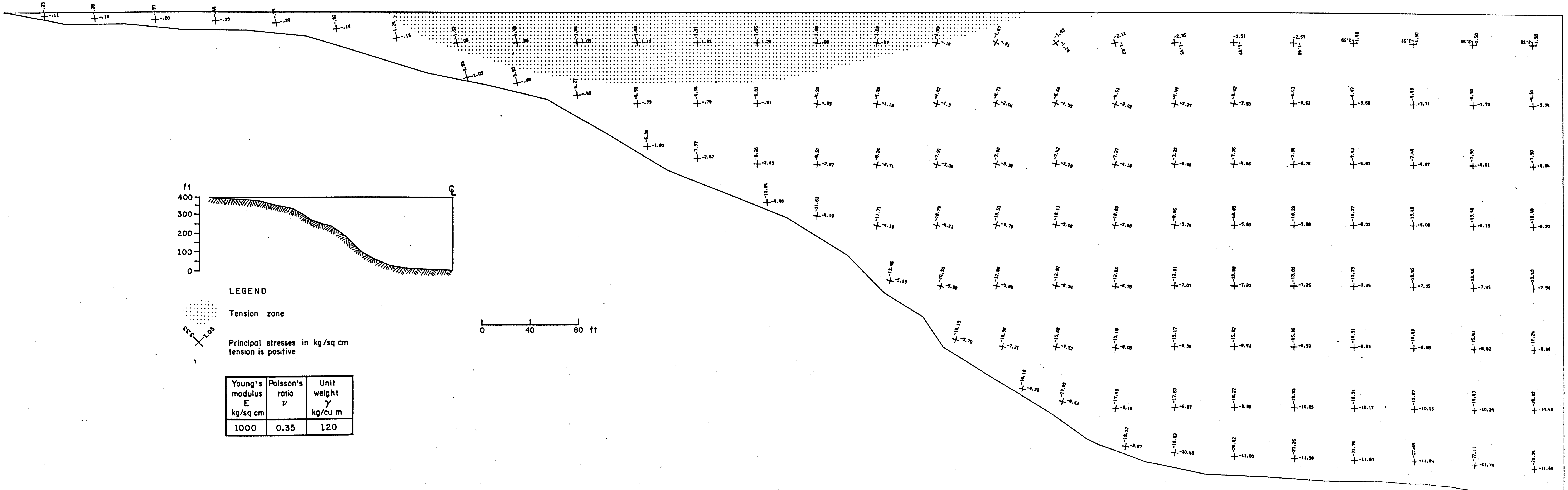


Fig 3b. Measured longitudinal strains along crest of Summersville Dam



Young's modulus E kg/sq cm	Poisson's ratio ν	Unit weight γ kg/cu m
1000	0.35	120

Fig 3a. Principal stresses in Summersville Dam

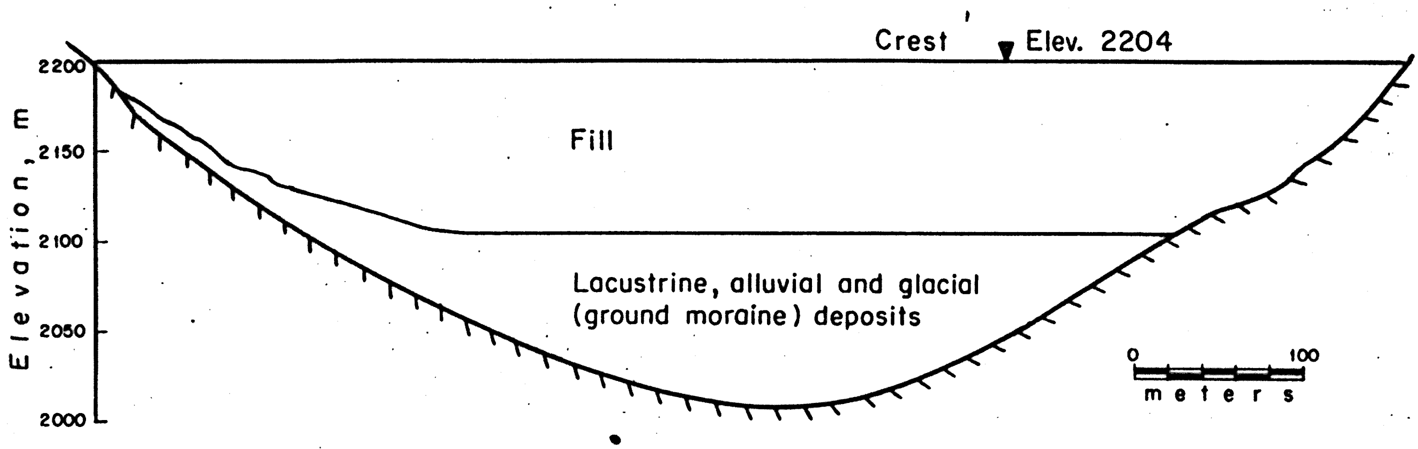


Fig 4a. Longitudinal profile of Mattmark Dam

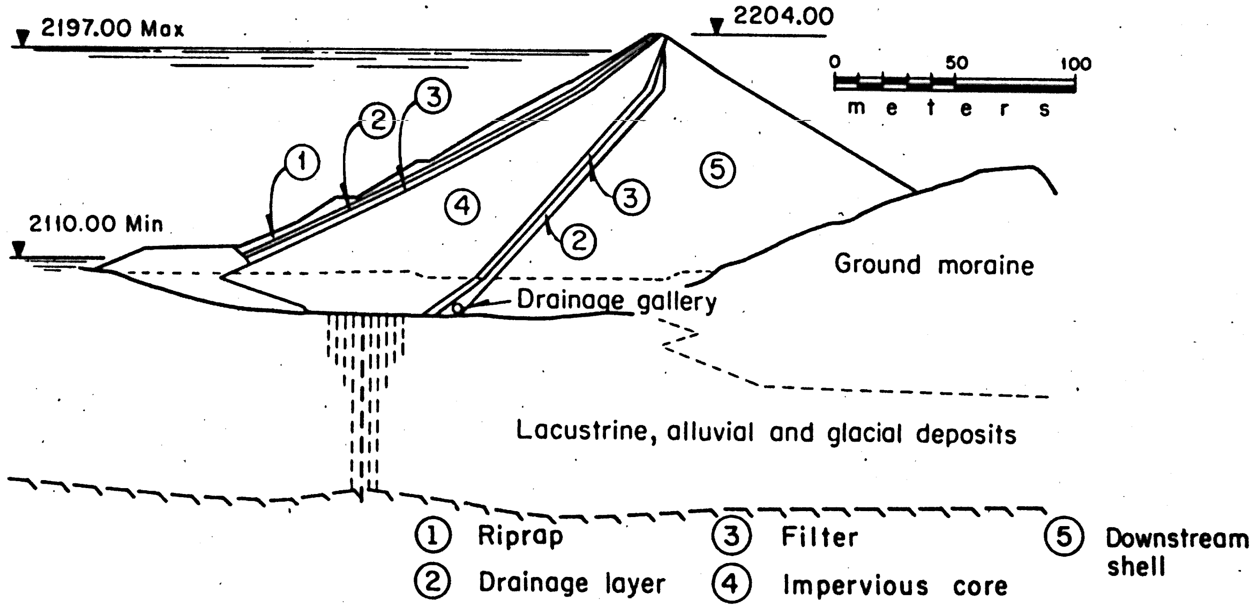


Fig 4b. Cross section of Mattmark Dam

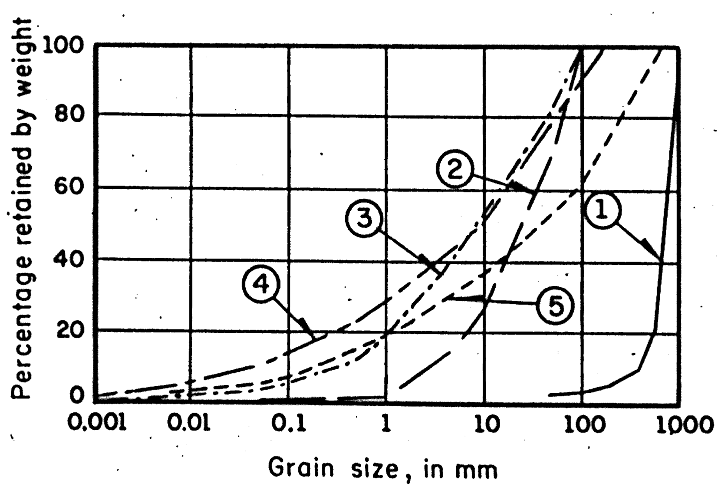


Fig 5. Grain size distribution of materials in Mattmark Dam

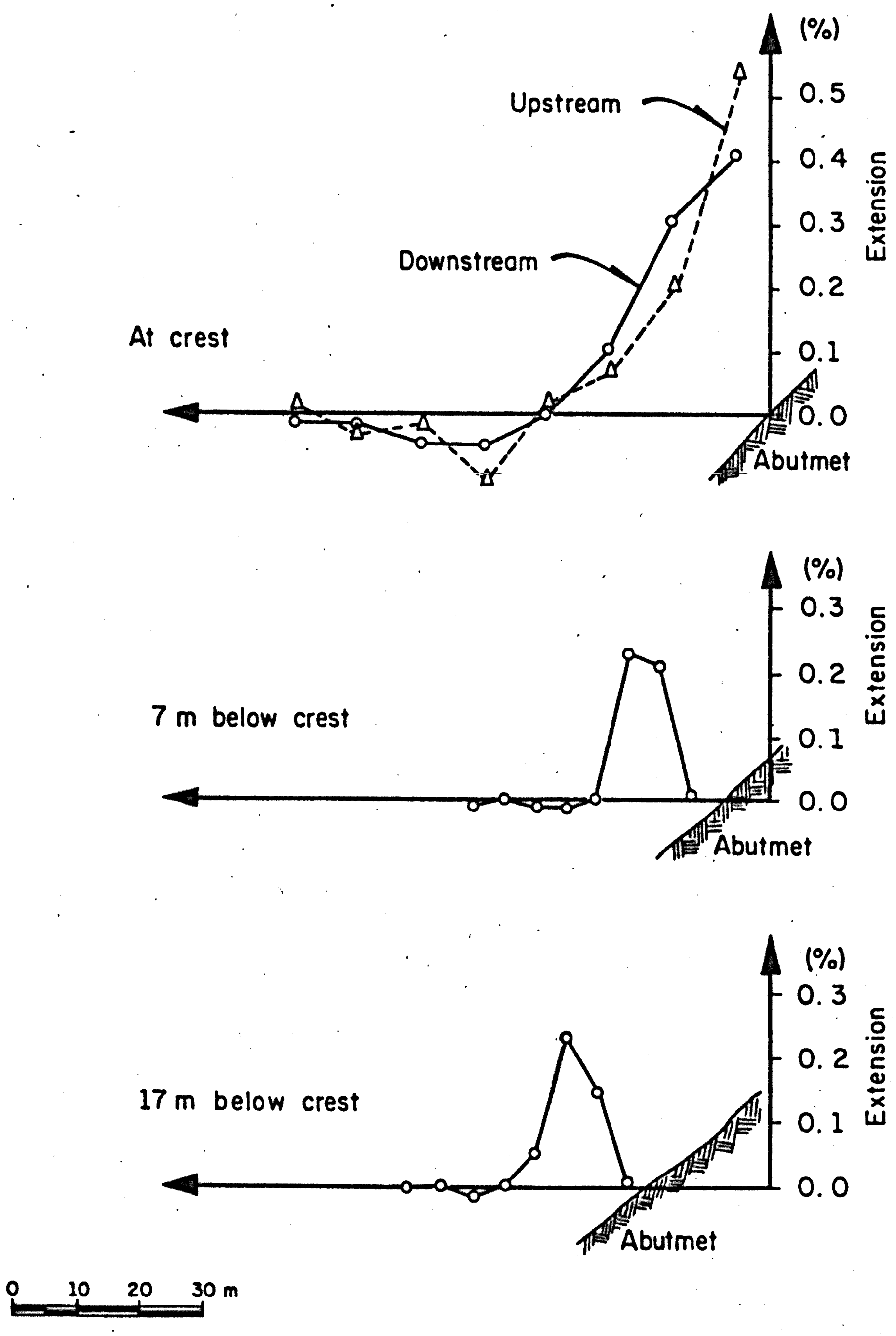


Fig 6. Measured horizontal strains along crest of Mattmark Dam

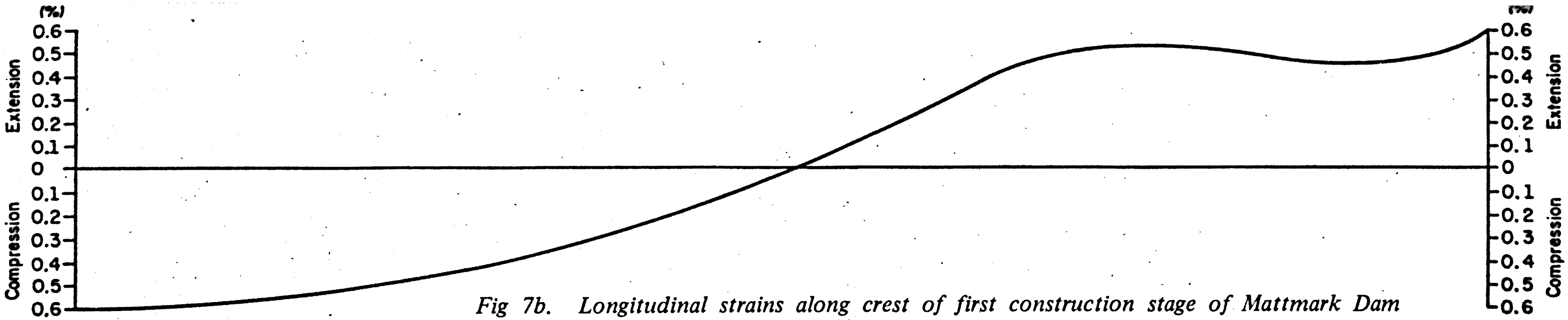
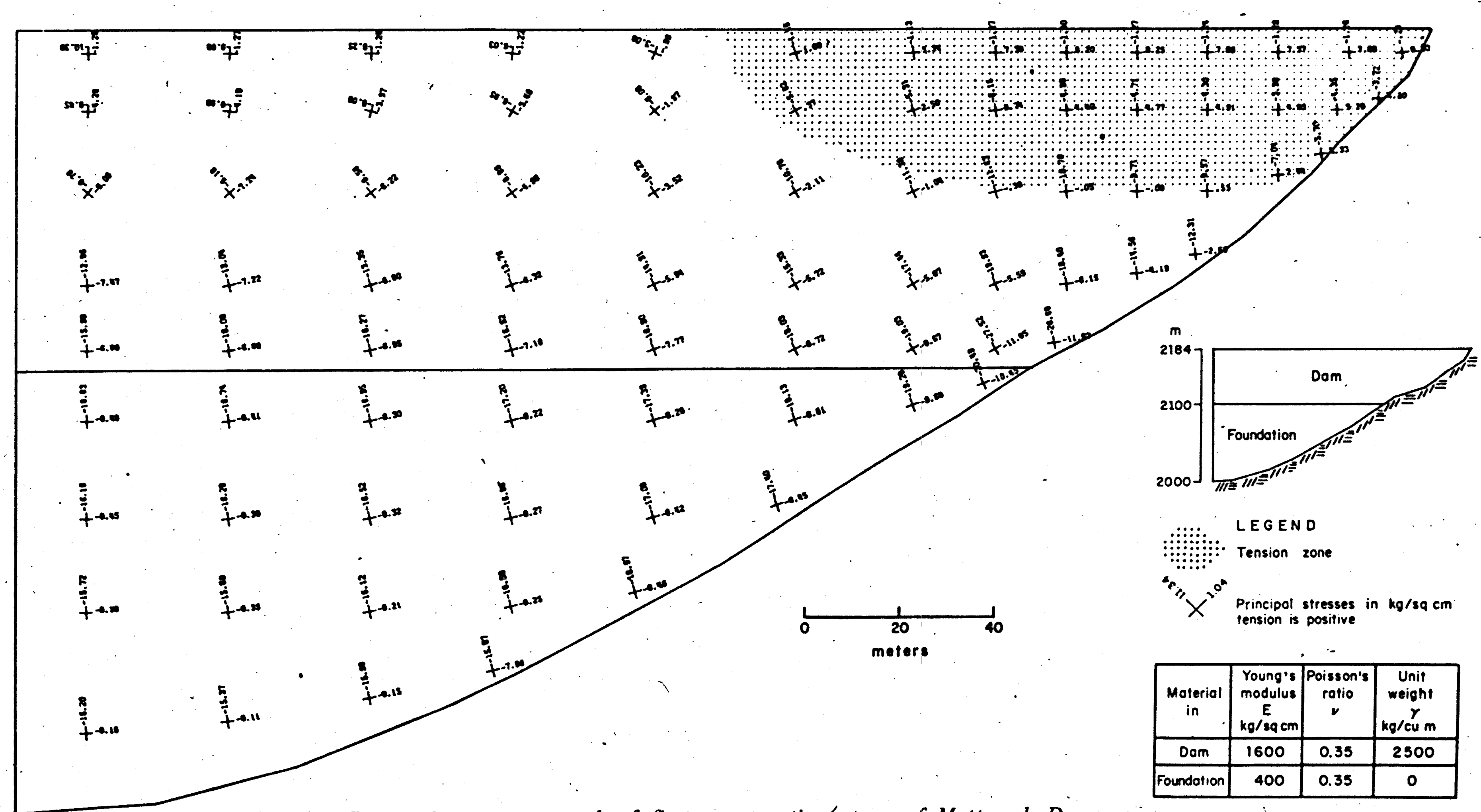
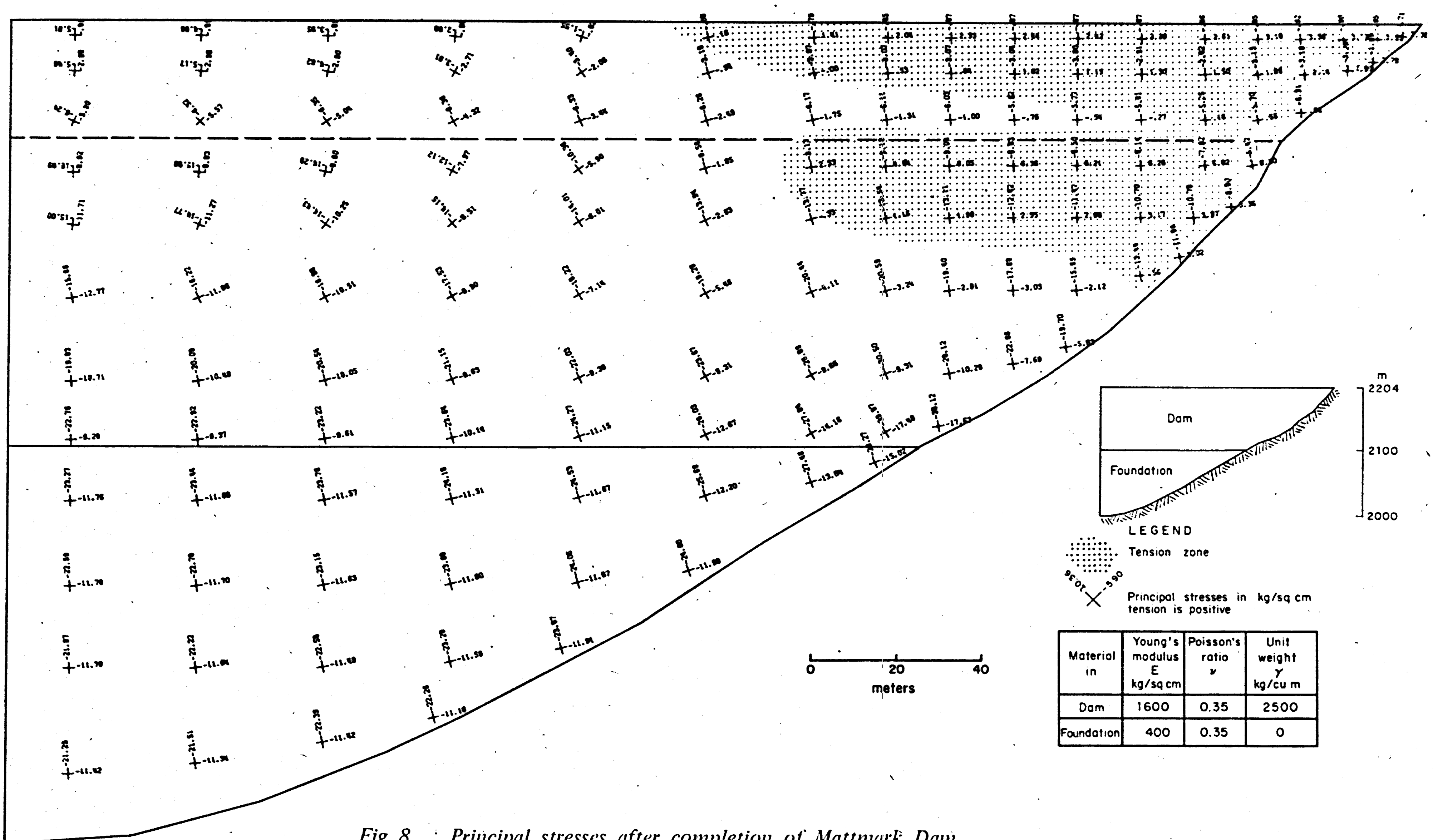


Fig 7b. Longitudinal strains along crest of first construction stage of Mattmark Dam



Material in	Young's modulus E kg/sq cm	Poisson's ratio ν	Unit weight γ kg/cu m
Dam	1600	0.35	2500
Foundation	400	0.35	0

Fig 7a. Principal stresses at end of first construction stage of Mattmark Dam



Material in	Young's modulus E kg/sq cm	Poisson's ratio ν	Unit weight γ kg/cu m
Dam	1600	0.35	2500
Foundation	400	0.35	0

Fig 8. Principal stresses after completion of Mattmark Dam

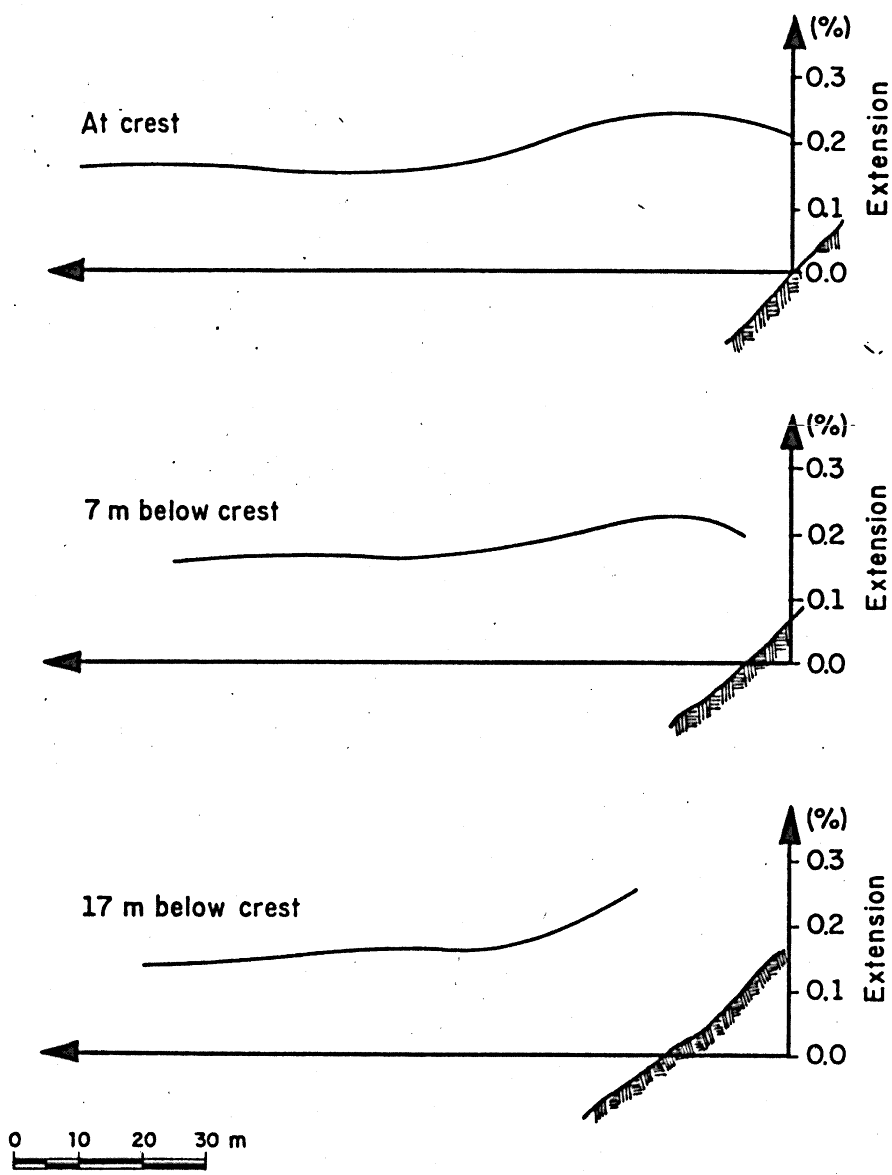


Fig 9. Computed horizontal strains in the top zone of Mattmark Dam

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13. ABSTRACT

The purpose of the investigation is to compare the longitudinal strains observed along the crests of Summersville and Mattmark Dams with the results of analyses using the finite element method. All materials were assumed to be linearly elastic, with equal properties in tension and compression. The only load considered was the weight of the embankment.

The results of the analyses show good agreement between the measured and the computed strains. Similar results were also obtained in previous investigations by the author. It is concluded that this method of analysis is a meaningful tool in designing earth and rockfill dams.

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