INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

SIMSG ISSMGE

*This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:*

*https://www.issmge.org/publications/online-library*

*This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.*

# Arching in the Foundations of a Large Dam

## Effet de Voûte dans la Fondation d'un Grand Barrage

G.LEFEBVRE Université de Sherbrooke,
O.DASCAL Hydro-Québec,
G.ST-ARNAUD Formerly Université de Sherbrooke, Québec, Canada

SYNOPSIS The Manicouagan 3 dam, located in norther Quebec, Canada, is a zone earth dam 110 m high founded on a sediment filled canyon 125 m deep. Seepage in the foundation is controlled by a positive cut-off consisting of interlocking cast-in-place concrete piles and panels. A finite element study has show that the weigh of the dam was transmitted by arching in the foundation soil to the walls of the steep canyon. Electrical extensometers in the cut-off wall and bedrock settlement points indicate that a significant cross-valley arching has developed during construction in the cut-off wall and in the foundation soil. Practical implications of arching effect on the cut-off wall are discussed.

### I INTRODUCTION

Two dimensional plane strain analyses provide an accurate estimate of deformations and stresses in centrally located transverse sections of a long dam. However the transverse sections near the center of a dam in a steep-walled valley cannot be considered in plane strain because of the effect of cross-valley arching. Lefebvre, Duncan and Wilson (1973) by comparing results of two dimensional plane strain analyses of the transverse section with those of three dimensional analyses have shown that cross-valley arching can influence considerably the deformation and stress condition for steep-walled valleys.

The main dam of the Manicouagan 3 Hydro Electric Project was commisioned in northern Quebec at the end of 1975. The embankment was built above a compressible foundation resting in a very steep-walled valley. The purpose of this paper is to present results of a finite element study and interpretations of instrumentation data during and after the construction of the dam which indicate a significant arching effect in the foundation.

### II DESCRIPTION OF DAM AND SITE

The Manicouagan 3 Hydro Power Development Project is located on the Manicouagan river some 88 km north of its confluence with the Saint Lawrence river and at about 560 km north east of Montreal, Quebec. At the site of the main embankment, 107 m high, the river valley narrows considerably and both banks rise steeply above its floor which is underlain by a 125 m deep alluvial canyon.

The general aspect of the valley at the site is shown on fig. 1. The pleistocene glaciations have molded the valley into a characteristic U shape with a deep V gorge extending approximately from El. 57 m to El. - 16 m. The rock slopes of the gorge average 65° while above the gorge up to the crest of the dam, rock slopes average 55° on the right bank and 40° on the left bank.

The bedrock is a coarse grained anorthosite of precambrian age with very little weathering at the surface. Sediments in the valley vary from fine silty sands to boulders rarely exceeding 1 meter in diameter. Generally, the boulders are located in the gorge and near bedrock with the sands and gravels in the center of the valley. Extensive zones of fine loose uniform sands were identified slightly below the riverbed. On the basis of punctual tests in boreholes, the permeability of the sediments was estimated between $1x10^{-2}$ and $1x10^{-1}$ cm/sec.

The cross section of the dam is presented on fig. 1. The central core is built of glacial till and protected by a large transition zones. The shoulders of the dam are sand and gravel and sloped at about 3:1. The cut-off wall with a maximum depth of 125 m consists of two diaphragms, 3 m apart and built of interlocking cast-in-place concrete piles 0.6 m in diameter. On each side of the valley where the alluvions become gradually thinner, the piles are replaced by cast-in-place concrete panels. Detailed information on the design and the site conditions have been presented by Dreville and al (1970).

### III FINITE ELEMENT ANALYSES

The analyses described herein were conducted before the construction of the embankment using finite element procedures for analysis of embankment developed by Kulhawy, Duncan and Seed (1969). The analyses simulated the built up of the embankment in layer and employed non-linear, stress-dependent, stress-strain relationships for the tangent modulus and the tangent Poisson's ratio of the embankment soils.

By comparing with results obtained by three dimensional finite element analyses, for a dam with 2.5:1 slopes, Lefebvre, Duncan and Wilson (1973) have shown that two dimensional plane strain analyses of the maximum longitudinal section can give fairly accurate representation of stress and strain in that section. Plane strain analyses of the maximum longitudinal section of Manicouagan 3 embankment should then gives even more accurate results due to the smaller average slope of the embankment (3:1).

The principal soil parameters used in the analyses are presented on Table 1 and have been derived from field and laboratory tests and from data presented by Kulhawy, Duncan and Seed (1969).

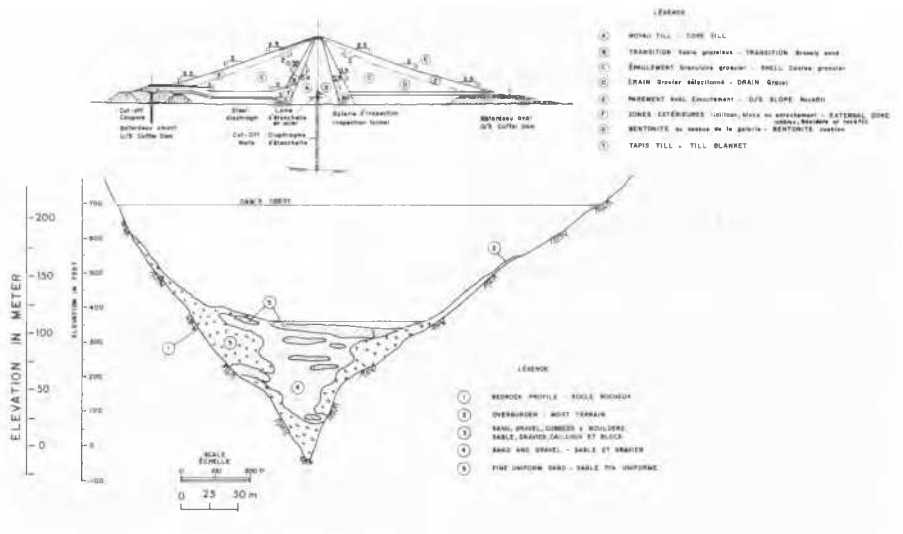


Fig. 1 General aspects of the valley and cross section of the dam

Table 1

| | Foundation sand | Foundation gravel | Dam Material |
|---|---|---|---|
| Effective density $\gamma'$ (kg/m$^3$) | 1003 | 1243 | 2242 |
| Cohesion c' | 0 | 0 | 0 |
| Friction angle $\Phi$ | 37° | 40° | 34° |
| Modulus number K | 350 | 715 | 525 |
| Modulus exponent n | 0.65 | .5 | .5 |
| Poisson's ratio parameters | | | |
| G | .47 | .50 | .38 |
| F | .21 | .27 | .12 |
| d | 2.1 | 7.2 | 5.3 |

The different parameters related to the modulus or the Poisson's ratio are well described in Kulhawy, Duncan and Seed (1969) or Duncan and Chang (1970).

In order to simulate the zone of loose sand identified in the upper part of the foundation, two zones were considered in the foundation, a zone of dense gravel in the lower part and a zone of loose sand with a lower modulus for the upper 37 meters. The properties of the core were assumed throughout the embankment in the analyses of the longitudinal section. This choice is of little consequence because the till of the core has deformation properties very similar to the gravel of the transitions and shoulders. Due to the very high modulus of the rock, the nodal points on the contour of the valley were assumed fixed. The initial stresses in the foundation were made equal to the overburden pressure.

FINITE ELEMENT RESULTS

Contours of major and minor principal stresses at th end of construction are shown on fig. 2. Some arching or beam effect are readily observed on the $\sigma_3$ contours. Because vertical displacement is larger in the center of the section, zones of low confining pressure develop close to the rock abutment. The phenomenon appears less significant at the top of th dam because the fill was added layer by layer. Arching in the foundation is evident in the $\sigma_1$ contour the major principal stresses increase in the dam fro top to bottom but decreases with depth in the founda tion.

Arching in the foundation is very evident when the vertical stress at the center of the section calcula ted at the end of construction by the finite element method is compared to the overburden pressure. Figure 3 compares the initial stresses in the founda tion with overburden stresses at end of construction and with stresses calculated by finite element. The vertical stresses increase due to the weight of the dam is about equal to the overburden at the foundation surface but decrease rapidly with depth in the foundation. In the deepest strata of the foundatior close to the bottom of the canyon, the influence of the weight of the dam is seen to be negligeable. Th weight of the dam is transmitted by arching on each side of the canyon.

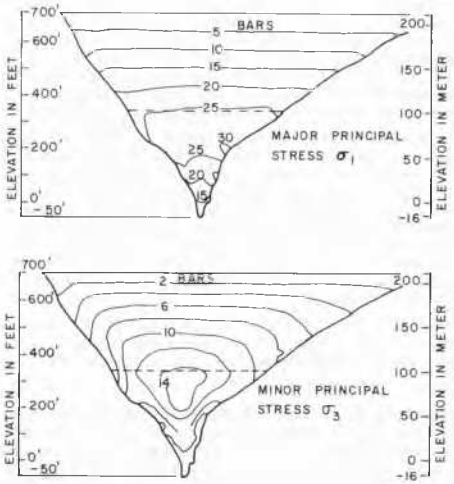


Fig. 2 Contours of major and minor principal stresses at the end of construction calculated by the finite element method

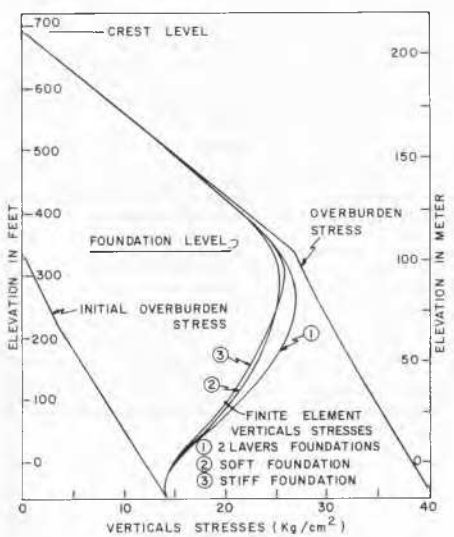


Fig. 3 Comparison of overburden stresses with vertical stresses calculated by finite element at the end of construction

With the soil parameters assumed in the analysis, the maximum settlement at the surface of the foundation was around 1.1 m. Two other analyses were run to check if variation of the compressibility of the foundation would have a great influence on the results. An analysis with a more compressible foundation was performed using the parameters assumed for the sand in all the foundation soils; a settlement of 1.4 m was obtained. A more rigid foundation was simulated by assuming that the whole foundation had the properties of the gravel; a maximum settlement of 0.8 m was thus calculated. The vertical stresses calculated in these analyses are compared on fig. 3 and are seen to be very similar to those obtained in the first analysis which assumed 2 layers in the foundation. Because the rock is assumed incompressible when compared to the soil, a small difference in the compressibility of the foundation does not change much the arching which develops in the foundation soil.

## IV EVIDENCE OF ARCHING FROM THE INSTRUMENTATION DATA

The development of the arching phenomenon in the foundation soil has been confirmed by the results of the instrumentation installed in the cut-off walls.

### INSTRUMENTATION LAYOUT

Due to the unusual problem caused by the deep permeable foundation, an extensive array of instrumentation was installed at the Manicouagan 3 main embankment. Only the instrumentation from which the arching phenomena can be appreciated, will be briefly described. Electrical extensometers have allowed the measurement of strain along the whole length of some piles and have given an idea of the stresses existing in the foundation soil. Settlement points set in the bedrock below the concrete cut-off wall have permitted reading of the bedrock compression during construction and provided indication of the stresses transmitted to the bedrock by the foundation soil. The readings from these instruments were recorded in an inspection tunnel sitting on the double cut-off wall.

### ELECTRICAL EXTENSOMETERS

The Carlson type extensometers, of 1.5 and 3.0 m length, were assembled in chains (up to 68 units) to cover the length of the pile in which they were installed. Ten piles were instrumented with electrical extensometers; only 2 representative sets of results will however be discussed in this paper.

### BEDROCK SETTLEMENT POINTS

The bedrock settlement points consist of a 2.5 cm diameter metal rod installed in a 10 cm diameter steel casing and anchored in the rock for at least 3 m. A dial gauge placed on the top of the rod in a recess of the tunnel's lining allows the measurement of the pile compression. Precise leveling on the top of the rods in the tunnel allow an evaluation of the bedrock settlement. Some of the instrumented piles are shown on fig. 4.

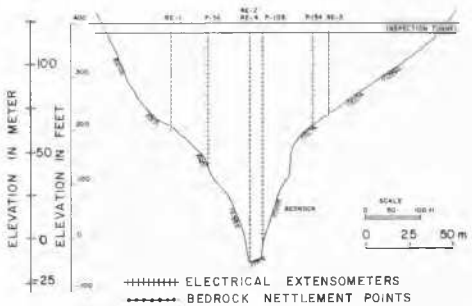


Fig. 4 Location of instrumentation piles

INTERPRETATION OF THE INSTRUMENTATION RECORDS

Electric extensometers

The large difference in compressibility between the foundation soil and the concrete cut-off wall has induced load concentrations on the walls, resulting in high compressive stresses or high compression strains in the piles and panels of the walls. The values of these stresses or strains are function of the wall-soil movement, the wall-soil friction and the normal pressure acting on the walls.

A fairly good idea of the compressive strain pattern in the wall and consequently of the wall-soil relative movement is given by the extensometers installed in the walls. The strain versus depth are presented for various stages of construction on figure 5 for a pile, 128 m. long, located at the center of the valley. On figure 6, the strain and the cumulative deformation at the end of construction are presented versus depth for a pile 55 m long and sitting on the right side of the valley above the deep V shape canyon. For the long pile located in the center of the valley (fig. 5), the strain in the early stages of construction is fairly constant through depth. As the embankment progressed, the strain distribution changed gradually; at the end of construction the compression strain increases from zero at the top of the pile to a maximum located at about mid-depth and then decreases gradually to zero at the tip of the pile.

In the upper portion of the foundation, the soil compresses more than the concrete pile and induces additional stresses in the pile by negative skin friction. In the lower half of the pile, the gradual decrease of the axial stresses is explained by two factors. While it has not been studied by finite element, it is believed that some cross-valley arching has developed in the cut-off wall and has resulted in a stress transfer from the center of the cut-off wall to the sides of the valley. In addition to that, very little or no additional stresses were transmitted to the pile by negative skin friction in that region due to a relatively small compression of the soil and to the small normal stress on the cut-off wall. In the last 80 feet of the pile where the stress decreases rapidly, it is also very possible that the concrete pile compressed more than the adjacent soil and transmitted stress by positive friction to the adjacent soil.

For the shorter pile resting on the edge of the deep V shaped canyon (fig. 6), the strain at the end of construction is seen to increase from zero at the top to high values in the lower portion of the pile. In the deeper quarter of the pile, high compressive strain have been recorded reaching values close to 1% at the bottom. Such a high strain for exceeds the strain at failure of the concrete. Plastic yield or compression failure indicated by the strain is also easily detected on the cumulative deformation curve where a sudden and large deformation is seen at the bottom.

The strain increasing rapidly with depth is an indication of large additional stresses being transmitted by friction to the pile. The soil along the pile is compressed more than the concrete of the wall and a state of high stresses existed to mobilize friction around the pile. The phenomena is more important along the lower part of the pile due to additional stresses being transmitted to that zone by the arching effect which has developed in the soil and in the cut-off wall.

The strain in a pile on the left edge of the deep canyon (P-56 on fig. 4) has also been recorded and show the same pattern as the pile discussed above.

BEDROCK SETTLEMENT POINTS

Settlement of the bedrock under the cut-off wall has been measured during construction at three different locations, (fig. 4) one at the bottom and one on each side of the deep canyon. Settlement of the bedrock is an indication of the stresses transmitted by the walls and by the foundation soil.

The deformations of the bedrock during construction are presented on fig. 7 as a function of time. If a correction for the zero reading is applied to the curves of fig. 7, the settlement of the bedrock a few months after completion is about 4 mm in the bottom of the canyon and 10 mm on each edge of the canyon. These readings seem to indicate a state of stresses appreciably higher on each side of the canyon than in the bottom, thus strongly advocating the hypothesis of cross-valley arching in the upper zone of the foundation soil and in the cut-off wall.

The interpretation of these two different types of measurements, strain in the concrete piles and bedrock displacements, clearly indicates that a significant arching effect has developed in the foundation. These measurements were however related to the behaviour of the cut-off wall and did not clearly show which part of the behaviour was due to arching in the cut-off wall itself or in the foundation soils. Due to the dimensions and nature of the wall, composed of interlocking vertical elements which cannot sustain large horizontal stress in the plane of the wall without large deformations, it is thought that cross-valley arching in the cut-off wall only cannot explain the observed behaviour and that an important arching effect had to developed in the upper portion of the foundation soil as predicted by the finite element studies.

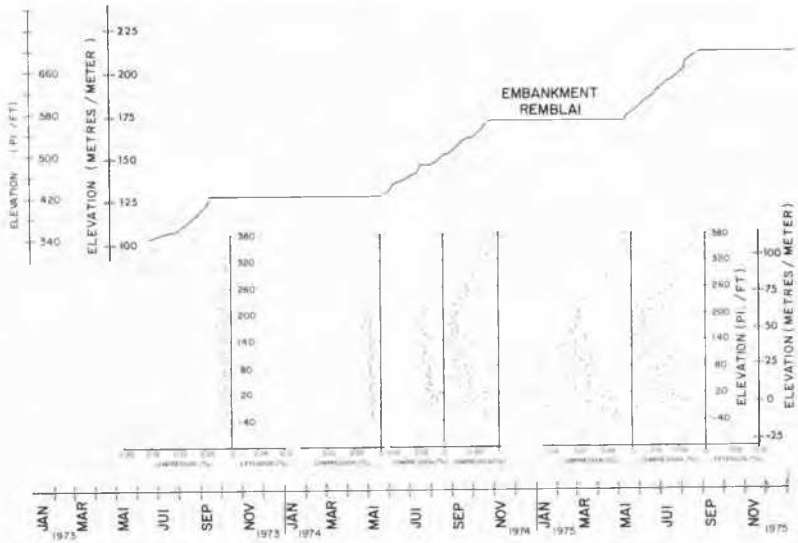


Fig. 5 Strain measured in a long pile at various stages of embankment construction

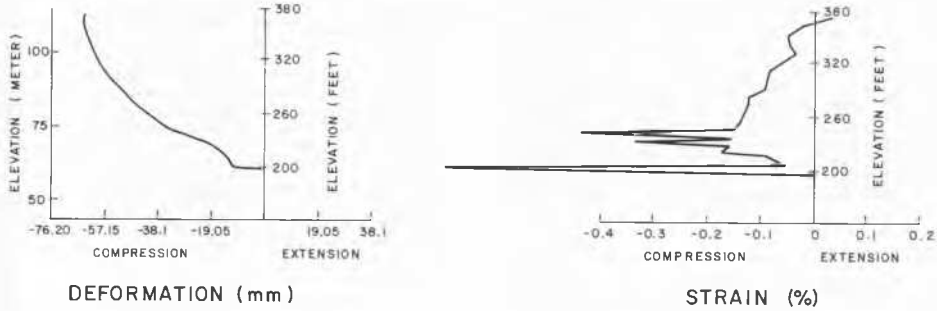


Fig. 6 Strain and cumulative deformation at the end of construction for a lateral pile

PRACTICAL IMPLICATIONS OF CROSS VALLEY ARCHING

'inite element studies as well as instrumentation re-
:ords indicate, that due to an arching effect in the
'oundation soil, the construction of the embankment
ıas induced very little additional vertical stresses
.n the lower zone of the foundation soil. On the
›ther hand, due to a load transfer by the soil arch,
:ones of vertical stresses, higher than the weight of
:he soil above, have developed on each side of the
/alley. It is evident that this particular state of
:tresses in the foundation soil has influenced the
behaviour of the rigid wall cutting through the foun-
dation soil.

STRESS DISTRIBUTION IN THE PILES

All the piles of the cut-off wall are embedded in sound bedrock and should then be considered as end bearing piles. In the long pile at the center of the valley, one should have expected axial stresses in the pile increasing gradually with depth due to negative skin friction down to almost the end of the pile. Under those conditions, the lower part of the pile would have to be designed to resist the largest axial stresses.

Due to cross valley arching, however, the long piles show small axial stresses at the bottom and the highest stresses at the middle.

MAGNITUDE OF STRESSES IN THE PILES

In the early stage of design, it was believed that due to negative skin friction the axial stresses will be appreciably higher in the long piles of the center of the valley, than in the short pile on the sides of the valley. The designer was then more concerned with possible compression failure in the long piles rather than in the short lateral piles.

Due to load transfer by cross-valley arching the axial

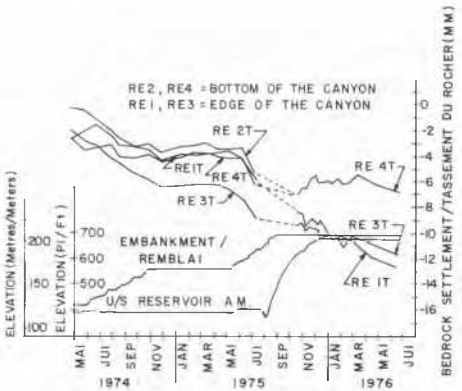


Fig. 7 Progression of bedrock settlements during construction

stresses were higher in the short pile. Large strain measured in the bottom end of the short pile indicated some failure or crushing due to the high compressive stresses.

EFFECTIVE STRESS IN THE FOUNDATION

Due to cross-valley arching, the construction of the dam has caused very little vertical stresses increase in the foundation soil at the bottom of the deep canyon. After completion of the embankment it may be assumed that the effective stresses in this zone was not appreciably higher than it was before construction. At the reservoir filling, the pore water pressure in the upstream zone of the foundation has increased, assuming a water tight cut-off wall, by a value close to full reservoir head. Rough calculations indicate that, under these conditions, the water pressure reaches values almost as large than the total vertical soil stresses and, according to the finite element studies, larger than the total horizontal soil stresses.

It is believed that these conditions may somehow decrease the effectiveness as a water barrier of the mud coating existing at the surface of the cast-in-place concrete elements of the cut-off wall.

DOWNSTREAM CONFINEMENT OF THE CUT-WALL

If the cut-off wall is assumed water-tight, the full head of the reservoir is acting along the whole depth of the upstream face of the wall. As illustrated by the finite element results of fig. 2 the stresses after construction decrease with depth in the foundation soil. In other words, the stresses which resist the downstream movements of the wall under the reservoir head decrease with depth. Inclinometers cast-in the cut-off wall have indicated a larger downstream movement of the walls at the bottom of the canyon during the filling of the reservoir.

CONCLUSIONS

The main embankment of the Manicouagan 3 Power Development was built in a narrow and steep-walled valley extending 125 m below the foundation of the dam. Finite element studies have shown that the full weight of the embankment was transmitted to the sides of the valley by arching in the foundation soil and that the increase in vertical stresses was very small in the lower half of the foundation soil.

Infiltration in the foundation soil had to be controlled by a positive cut-off wall which was extensively instrumented. The strain in piles obtained by electrical extensometer and the compression in the bedrock read from settlement points clearly show that an important arching effect has developed in the foundation.

ACKNOWLEDGMENTS

The finite element analyses presented in this paper were part of a study done at the University of Sherbrooke and subsidized by Hydro-Quebec. The Manicouagan 3 Main Dam was designed by Asselin, Benoît, Boucher, Ducharme, Lapointe Inc., consultants, in cooperation with the engineering staff of Hydro-Quebec and with the active participation of a Board of Consultants constituted of Professor A. Casagrande, Chairman, Mr. F.B. Slichter and the late Dr. F.A. Nickell and Mr. R. Peterson. Mr. J.D. Wilson acted as consultant on instrumentation.

The authors wish to thank Hydro-Quebec for permission to publish the data presented herein.

REFERENCES

Dreville, F., Paré, J.J., Capelle, J.F., Dascal, O. and Larocque, G.S., "Diaphragme en béton moulé pour l'étanchéité des fondations du barrage Manicouagan 3", Dixième Congrès des grands barrages, Montréal.

Duncan, J.M. and Chang, C.Y., 1970. "Non Linear Analysis of Stress and Strain in Soils", Journal of the Soil Mechanics and Foundations Division, ASCE, vol. 96, no. SM5.

Kulhawy, F.H., Duncan, J.M. and Seed, H.B., 1969. "Finite Element Analyses of Stresses and Movements in Embankments During Construction", report no. TE-69-4, Office of Research Services, University of California, Berkeley.

Lefebvre, G., Duncan, J.M. and Wilson, E.L., 1973, "Three Dimensional Finite Element Analyses of Dams", Journal of the Soil Mechanics and Foundations Division, ASCE, vol. 99, no. SM 7.