The Use of Fat Clay in Dam Construction in Israel

L'Emploi des Argiles Grasses dans la Construction des Barrages en Israël

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Summary

In many parts of Israel the scarcity of granular material has led to the necessity of building homogeneous dams of fat clay. Several small dams have been constructed since 1952, and been a source of interesting and useful data on the construction and performance of dams built from plastic soils. The experience gained to date is being utilized for planning of larger structures now in the design stage. The difficulties of evaluating shear strength for this type of soil are stressed. Dam sections already built are presented and design features discussed, including slope protection and drainage provisions.

Problems peculiar to the clay during construction are mentioned. Results of field pore pressure and settlement observations are given. Because of the swelling nature of the clay a rise, rather than settlement of embankments, has been found at the smaller sections. Post-construction performance is reported in the light of several years service.

Sommaire

La rareté des matériaux granuleux dans de nombreuses régions d'Israël a conduit à la nécessité de construire des barrages homogènes en argile grasse. Plusieurs petits barrages ont déjà été construits depuis 1952 et ont fourni des données intéressantes et utiles sur la construction et le comportement des barrages en terre plastique. L'expérience acquise jusqu'à ce jour est utilisée pour l'étude des constructions d'envergure plus grande projetées actuellement. Les difficultés d'évaluation de la résistance au cisaillement pour ce type de terrain sont soulignées. Des sections de barrages achevés sont présentées et les particularités de construction discutées y compris les mesures de protection des pentes et les dispositifs de drainage.

Les problèmes particuliers attachés à l'usage de l'argile pendant la construction sont mentionnés. Les résultats d'observations sur la pression de l'eau interstitielle et le tassement sont présentés. A cause du gonflement de l'argile, une expansion, au lieu de tassement, du remblais a été constatée dans les petites sections. Les données sur le comportement des barrages en terre après leur mise en service sont basées sur plusieurs années d'observations.

Introduction

The urgent demand of the Israel economy for provision of water storage facilities has made it necessary in some cases to select reservoir sites which are far from ideal. A number of the proposed earth dams are situated in areas where there is a scarcity or absence of granular materials suitable for incorporation in the embankment. The native soil is often found to be a highly plastic fat clay. Since the obtaining of granular soils or crushed rock involves unusually high transportation or quarrying costs, economy demands maximum utilization of the local clay, resulting in designs of essentially homogeneous embankments with minimum shells and quantities of drain materials. These clays are characterized by strong swelling properties and loss of strength upon saturation.

The experience in Israel of using fat clays for dams is considered of particular interest as having included a serious study of the problems involved in the design and construction of homogeneous clay dams, the application of engineering construction control and the accumulation of data on the performance of the dams already built.

Dams Constructed

Three fat clay dams have so far been constructed in Israel (Fig. 1). The first dam was built in 1952 in the Beit Netufa valley. It is about 11 m high and the total fill volume is 120,000 m³. The purpose of this small dam was not only to

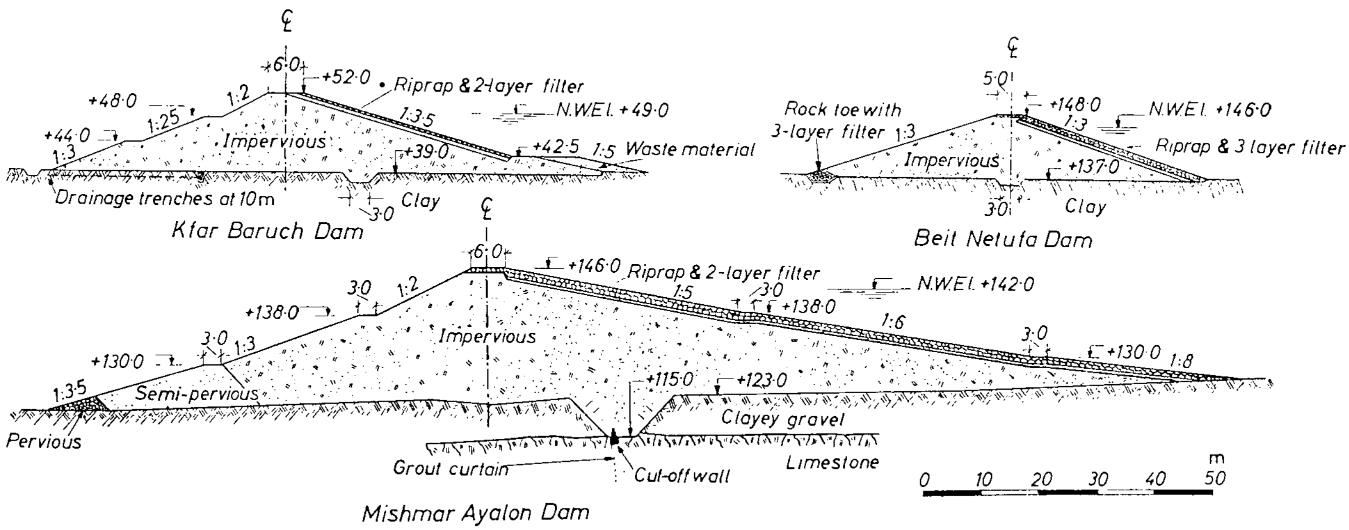

Fig. 1 Cross-sections of dams
Sections des barrages

store the winter run-off but to gain experience in the construction procedures for larger clay dams to be built in this area.

In 1953 the Kfar Baruch dam was built across the Kishon river. The dam is 13 m high and 355,000 m^3 in volume. It is intended to raise this dam in the future by 3·5 m at which time the total fill volume will become 700,000 m^3.

The embankment of the Mishmar Ayalon dam was constructed during 1953–54. The dam is 24 m high and the volume of fill is 450,000 m^3. In the future this dam may be raised to 32 m, resulting in a 900,000 m^3 fill.

Properties of the Clay

Physical properties—The material consists of 60 to 80 per cent clay of sizes ($< 5\,\mu$), 15 to 25 per cent silt of sizes 5 to 74 μ and 5 to 15 per cent sand and gravel. Its specific gravity is 2·74. The consistency limits are as follows:

Liquid limit:	71 to 95 per cent (average 82 per cent)
Plastic limit:	24 to 38 per cent (average 31 per cent)
Plasticity index:	39 to 67 per cent (average 51 per cent)
Shrinkage limit:	5 to 12 per cent (average 10 per cent)
Shrinkage ratio:	2·1 per cent

This material is classified as a highly plastic clay (CH); it is dark brown, of medium consistency, heavily slickensided, and without significant organic matter content. Petrographically it belongs to the Ca-Montmorillonite clays with 40 to 80 per cent montmorillonite in the $< 2\,\mu$ sizes. The cation exchange capacity is about 100 milliequivalents per 100 g of the clay fraction.

Engineering Properties

Density and moisture—In the natural foundation soils, at depths greater than about 1 m, the range of natural densities was 1250 to 1600 kg/m^3 where the highest densities were associated with the presence of sand and gravel. The relatively high densities of non-gravelly clays are attributed to the action of drying forces in the previous history of the clay. The natural moisture contents range from 20 per cent in the upper layers to 37 per cent at depth of about 8 m.

The maximum density of the remoulded material, compacted according to standard Proctor test, was found within the range of 1310 to 1440 kg/m^3 and the optimum moisture varied from 30 to 35 per cent.

Shear strength—Three general methods of evaluating shear strength were used in the testing programmes: undrained, drained and consolidated undrained: both triaxial and direct shear testing were employed. The following values are considered as representative for use in design:

Test method	c kg/cm^2	ϕ degrees
Undrained		
Unsaturated foundation clays	0·3	10
Saturated foundation clays:		
Pressures up to 0·7 kg/cm^2	—	23
Pressures over 0·7 kg/cm^2	0·3	0
Drained		
Unsaturated foundation clays	0·6	15
Saturated foundation clays	0·2	15
Unsaturated embankment material (maximum density at optimum moisture)	0·8	10
Saturated embankment material	0·2	15
Consolidated undrained		
Saturated foundation clay	0·2	10

For larger structures, where a pore pressure analysis is justified, conditions prevailing during construction, steady seepage, and sudden draw-down are analysed using the drained strength as a basis. For smaller structures the strengths available are taken directly from undrained test results. It is felt that the strength values obtained from tests made to date are still far from final in view of many difficulties encountered in the determination of pore pressures—effect of rate of shear on test results, the question of proper time necessary for consolidation, and similar problems peculiar to shear strength testing of clays. Basic research is being undertaken on the study of intergranular and pore pressures, strength under sustained loads, effect of swelling on shear resistance, and related phenomena.

Consolidation—Time-compression curves often did not follow the theoretical functions given by the Terzaghi theory. Such behaviour has been noted for pre-consolidated materials as well as for compacted soils and indicates that conventional time-settlement analysis methods may not be applicable. The one day period utilized between loading increments may not be allowing sufficient time for full relief of internal pore pressures and completion of primary consolidation. For illustration, typical consolidation data are presented:

Type of material	*Consolidation* per cent	*Load* kg/cm^2
	3	3·5
Foundation	6	7
	10	14
	4	3·5
Embankment	10	7
	16	14

Permeability—The permeability coefficient k_w for undisturbed soil varied between 10^{-5} and 10^{-8} cm/sec for samples taken down to 8 m depth. The higher values should be attributed to the presence of fissures and slickensides in many of the samples tested. Saturation itself does not lead to a complete closure of the cracks, but it was noted that the permeability became very low in those tests where confining pressures exceeded 1 to 2 kg/cm^2. The permeability coefficient for remoulded material was found to be 10^{-7} to 10^{-8} cm/sec or smaller. No data are yet available on the relative horizontal and vertical permeability of the clay.

Pore pressure—Results of pore pressure measurements obtained during triaxial testing were of the same order as those calculated from consolidation tests. Although considerable variations in pressure shown by individual specimens resulted from small differences in density and placement moisture, the test data indicate that for normal placement conditions a pore pressure of about one-third the fill pressure may be developed for an embankment of 25 m height and two-thirds of the fill pressure for an embankment of 50 m height.

Expansion—The amount of expansion and the pressure developed due to wetting was determined in standard type consolidometers for both foundation and embankment material. The maximum amount of expansion under no load was found ranging from 3 to 12 per cent of original thickness, while the 'swelling pressures' for zero expansion were found to vary between 0·5 and 2·0 kg/cm^2. The values depended on the initial placement moisture and density conditions.

Design

Stability—Stability studies were made according to the slip-circle method, analysing conditions for the construction, sudden draw-down and steady seepage states. Horizontal earthquake forces equalling 10 per cent of the weight of the dam were included in the analysis.

In most designs the eventual raising of the dams had to be taken into account since overall planning considerations made it necessary to increase the reservoir capacities in stages. From the point of view of design and construction, building in stages presents many difficulties, since it leads to the elimination of upstream borrow areas, necessitates the use of flatter slopes and special drainage provisions, and requires the stripping of the downsteam slope when construction is continued.

In order to provide a guide for future homogeneous, fat clay dam designs some general stability studies for dams up to 23 m height were made. The main conclusions reached in these studies were:

(1) The stability of the dams depends mainly upon the cohesion of the material since the angle of friction of the clay is very low. Accordingly, since shearing resistance

$$\tau_f = c' + (\sigma' - u) \tan \phi'$$

pore pressure, even if high, does not greatly influence the overall stability.

(2) The upstream slope, when the only surcharge on top of the compacted fill is a relatively thin riprap and filter layer, should not be steeper than 1:3 and preferably 1:3½, since shear strength is markedly reduced upon saturation at loads lower than 0·5 to 0·75 kg/cm^2 which leads to the danger of shallow slides at the point of contact between the clay and the riprap. It is therefore desirable to place a surcharge loading of semi-pervious or pervious materials on top of the clay at the upstream slope.

(3) The depth of the clay foundation is a very important factor for establishing the slopes of the dam. Dams over 12 m height on deep clay foundations require very flat slopes at about 10 m below the crest. Since the main purpose of this wider base is to weigh down the foundation, it may consist in its outer section of miscellaneous waste materials. Dams on shallow clay, gravelly or rock foundations, may have steeper slopes.

(4) The upstream slope is established by the sudden draw-down state analysis and the downstream slope by the steady state analysis. Saturated foundations lead to high construction pore pressure and in some cases may dictate flatter dam slopes. In some cases analyses, including the earthquake forces, govern the design in spite of a lower allowable factor of safety and a 50 per cent increase in cohesive strength assumption.

Factors of safety of 1·5, and, when earthquake forces are included, of 1·25 were considered adequate, since conservative shear strength values were used. Five stable dam sections obtained from these studies are shown schematically in Fig. 2.

Drainage features—The shortage of pervious soils and the need to take into account the enlargement of the dams in the future have been the main factors governing the drainage design. The small rock toe of the Beit Netufa dam (Fig. 1) is limited to the wadi bed since a larger dam was to be built shortly afterwards. Additional drainage provisions may be required, especially at the right abutment where ground water has appeared at the toe.

The trench system at Kfar Baruch has not proved to be very satisfactory. It seems that for these fat clays a continuous filter blanket is preferable to a trench system or a rock toe, since, in addition to moving the phreatic line towards the interior of the dam, it provides improved drainage for the foundation. For small dams on non-saturated foundations rock toes may be adequate.

Slope protection—The downstream slopes of the Beit Netufa and the Mishmar Ayalon dams have been seeded with Kikuyu grass, while at Kfar Baruch the downstream slope was left unprotected. Since there is no rain in Israel during the summer, sprinkling is required. No erosion has taken place on seeded slopes. Gully erosion has occurred on all unprotected slopes, in some cases to a depth of 1·3 m; cracks due to drying out during the summer have also developed on all unprotected slopes. In the future it is intended to cover the downstream slopes with coarse, erosion resistant materials. Where such a material is locally unavailable and seeding will be done, a 30 cm top-soil layer would be used since seeding on sterile soil, even with the addition of fertilizers, has resulted in a thin and

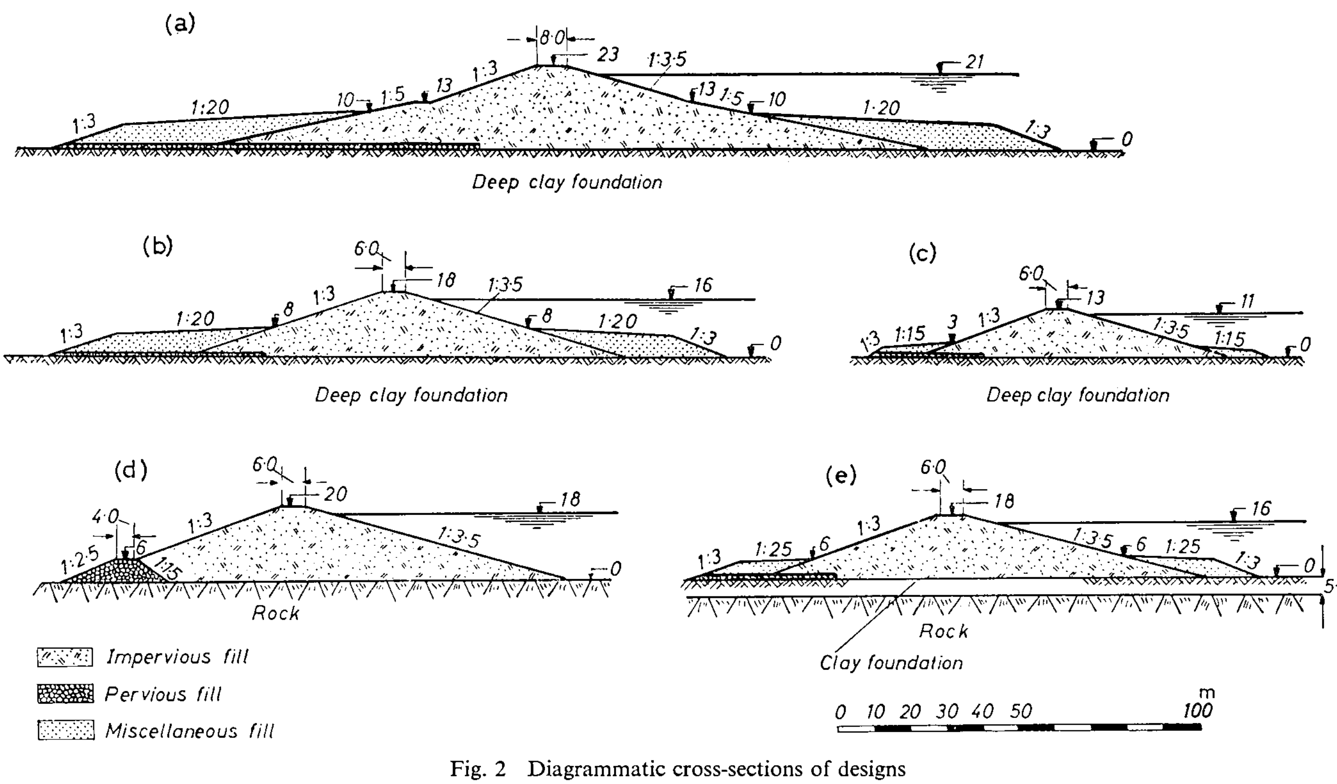

Fig. 2 Diagrammatic cross-sections of designs
Sections projetées schematiques

patchy grass cover. Attempts are being made to find a grass which would make sprinkling unnecessary.

The upstream slope at Beit Netufa is protected by a 1 m riprap layer on top of a 3-layer, 45-cm thick filter. At Kfar Baruch 40 cm of hand-placed riprap and at Mishmar Ayalon 80 cm of dumped riprap were placed on top of a 45-cm thick, 2-layer, filter. The maximum size of the boulders in all cases was about 40 cm. This type of protection has given good service.

In recent filter designs sand layers have been omitted and only two layers of coarse aggregate have been provided since advantage can be taken of the cohesiveness of the clay. Consideration to one-layer graded filters is being given.

Construction

Conventional methods of construction revealed certain difficulties peculiar to the fat clays. Excavation by scrapers in layers thicker than about 10 cm resulted in large lumps of soil which were difficult to break up and compact because of the high cohesion at the prevailing moisture contents. Less clods were obtained when excavation was carried out by belt loaders, and future construction methods will be aimed at use of such equipment.

It was desired to obtain moisture at or slightly below the optimum by the standard Proctor test. In most of the borrow areas the natural moisture was suitable once the areas were stripped. Difficulties arose from the drying of the clay surface during interruptions in construction, and in the encountering of material with excessive moisture at depths approaching ground water level. Since it was practically impossible to pulverize these clay lumps with ordinary discing or harrowing equipment, it was not feasible to adjust moisture content on the fill by mixing in water or air-drying the fill by mixing action. Therefore materials were selected as far as possible ahead on the basis of preliminary moisture data developed in borrow exploration. Sprinkling by tank trucks on the fill has been found necessary and is used to prevent crust formation in the previous fill layer before placement of new soil. It is hoped that better control will be obtained in future work with the use of more effective mixing equipment such as rotary hoes or tillers.

Compaction was carried out by sheepsfoot rollers, of the U.S. Bureau of Reclamation type, utilizing foot pressures of about 30 kg/cm^2. Compaction in excess of 97 per cent standard was ordinarily obtained with 10 to 12 roller passes on a layer of about 20 cm loose thickness. One difficulty arose from the clay lumps forming open voids and bridging over the depressions left by the roller feet. When construction traffic was not kept off the newly spread fill a compacted surface layer was formed which prevented proper roller penetration and compaction of the full layer depth.

Density tests for construction control were made by the sand volume of hole method, at the rate of one test per 1000 m^3. It was found impracticable to utilize the Proctor penetration needle for either moisture or density control because the presence of clay lumps produced erratic penetration readings. The difficulty of pulverizing the soil also prevented checking the maximum density of all the clay borrow materials, since excessive time was involved in preparing material to obtain a complete compaction curve. Moisture contents obtained by rapid drying on a hot sand bath were slightly higher than the usual standard of a 24-hour drying period of 105° C.

Dam Performance Observations

Piezometer installations have included the use of open-end, ½ in. tubes and porous piezometer tips (U.S.B.R. type). Observation wells, consisting of 2 in. perforated pipe, installed in 8 in. auger holes backfilled with gravel, have also been used along the toes of Beit Netufa and Kfar Baruch dams. The open-tube type of piezometers and observation wells have proved unsatisfactory because of the long lag caused by the low permeability of the clay. However, the porous cell type of piezometer has appeared to give more consistent and reliable results. Typical results of piezometer observations are shown for Kfar Baruch in Fig. 3. The main body of the embankment has not yet become saturated by flow from the reservoir, but water from the foundation is tending to rise directly into the fill. Fields downstream of this dam were found to be swampy from high ground water. In the Beit Netufa embankment, water heads above the toe were observed as a result of the action of springs in one abutment. Positive pore pressures due to compaction and consolidation of these dams have not been observed, which is attributed to low embankment heights and to the tendency of the clay to swell after compaction.

Settlement installations were generally similar to those of the U.S. Bureau of Reclamation, with pipe cross-arms at about 1½ m intervals of depth, fitted into telescoping vertical pipe sections. In addition, surface markers were used on the outside of the embankments to provide a measure of horizontal and vertical movements. Observations showed a large part of the settlement to have occurred during the construction period, mostly in the foundation. Embankment consolidation of Beit Netufa dam (Fig. 4) is typical, showing for a 9 m height that the lower third consolidated close to 2 per cent and is still compressing very slowly, while the upper third of the embankment showed no long-time consolidation effects after an initial compression of the order of 0·5 per cent. In fact, an interesting feature to note in the observations is that the upper layer of the embankment shows both swelling and shrinkage movements corresponding to the wet and dry seasons. In the Kfar Baruch dam, the swelling of the upper material was sufficient to compensate for the consolidation at the maximum section, while the abutment sections are actually as much as 3 cm higher than after construction. It should be also noted that horizontal

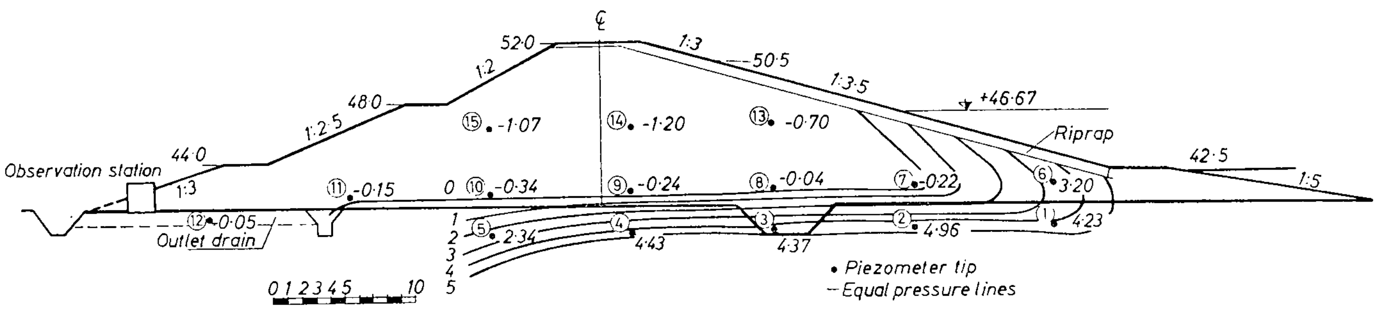

Fig. 3 Kfar Baruch dam—piezometer observations
Barrage de Kfar Baruch—observations piezometriques

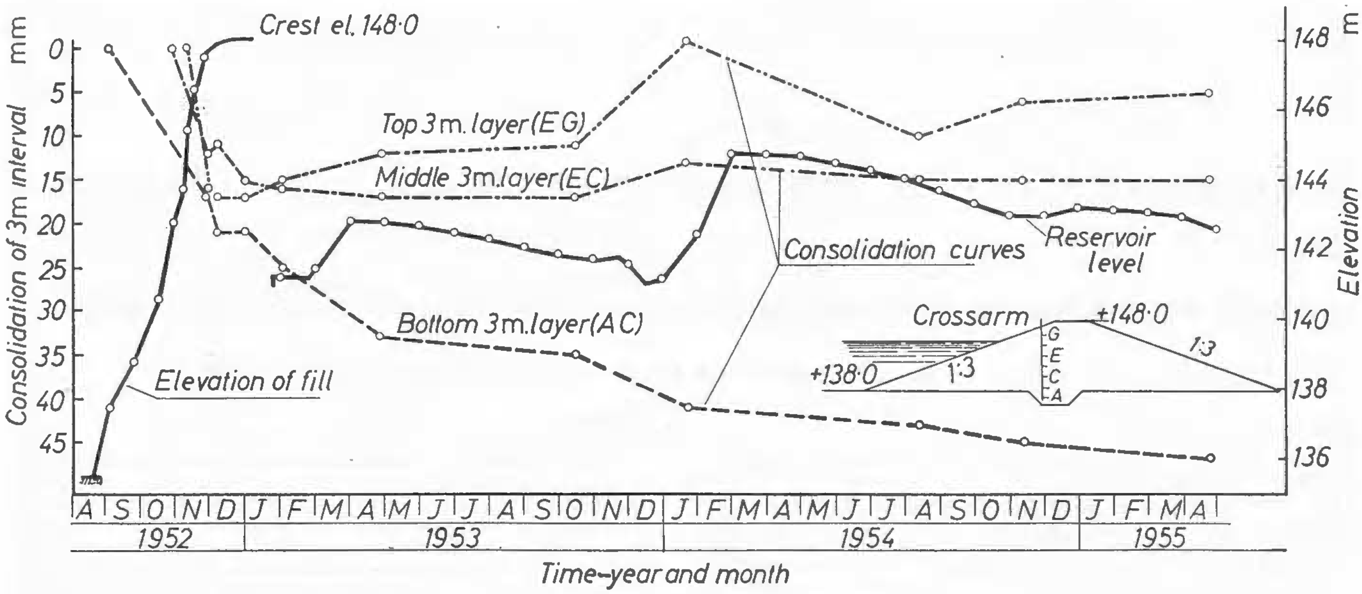

Fig. 4 Beit Netufa dam—settlement observations
Barrage de Beit Netufa—observations du tassement

movements of the downstream slope away from the reservoir have reached as much as 17 cm, which is attributed to swelling of the clay as well as some tendency for 'creep' of the slope.

The design and construction of the dams have been the responsibility of Water Planning for Israel Ltd. The work has been performed under the direction of Mr A. de Leeuw, Chief Engineer of the Reservoir and Research Division.

This paper is based on an internal report 'The use of Fat Clay for Dam Construction' by a study group which included the authors and Mr A. Yevnin of Water Planning for Israel Ltd.