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Wadi Qattarah Dams Case History

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SUMMARY

Two earthfill dams were constructed on the Wadi Qattarah in Libya between 1969 and 1972 but the filling was never achieved for both dams. After a partial filling well below normal water level, Secondary Dam failed in December 1977. Several weaknesses may be found in the features of both dams and have been analyzed by the authors on the basis of dam instrumentation results, measurements and observations of the Secondary Dam failure. Several possible causes of failure of this dam are presented. No one can however be taken as certain. The series of events as reported by eye witnesses appear to point out a typical phenomenom of piping, but the responsible mechanism can be attributed to various causes.

1. HISTORICAL INTRODUCTION

The protection of Benghazi, the chief town of Cyrenaica and second largest city in Libya, and the surrounding plain against flooding by Wadi Qattarah has long been a subject of studies which materialized in 1965-67 with the issuing the Wadi Qattarah Project (Fig. 1).

Fig. 1 - General Location of the Project

The upstream portion of the Project was formed mainly by the Main Dam and the Secondary Dam some 10 km farther downstream which were built from 1969 to 1972. The first filling started in May 1972 but the reservoir levels very seldom exceeded a third of the head and never reached the normal water level (NWL) at either dam. On 21 December 1977, the Secondary Dam failed with a water level well below NWL but fortunately major damage was avoided by a rather slow rate of retrogressive erosion of the embankment (about 6 hours) and by the relatively moderated volume of water stored, some 3.5 x 10^6 m^3 (Fig. 2).

Main Dam embankment was continuously monitored and two series of geotechnical investigations were carried out in 1978 and 1979 to ascertain that conditions similar to those at Secondary Dam did not develop.

In this paper the authors will describe the main geological and geotechnical features of both dams and report the observations made during the Secondary Dam Failure. On the basis of these features, observations and dams instrumentation results and measurements, they will present what are in their opinion the main weaknesses of these structures and they will analyze the possible causes of failure of Secondary Dam pointing out the most likely ones.

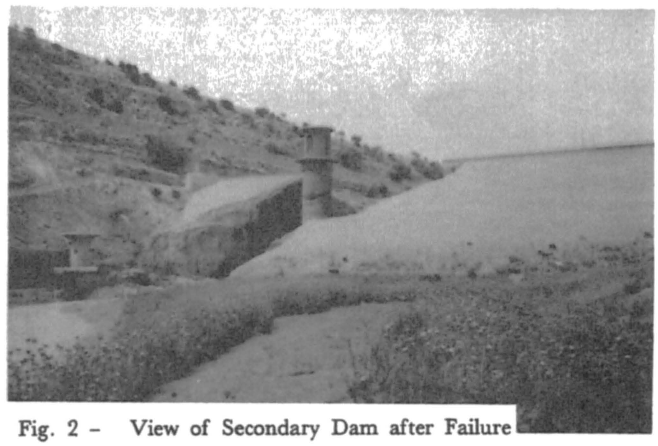

Fig. 2 - View of Secondary Dam after Failure

The design work for rehabilitation of the Wadi Qattarah scheme including the strengthening of the existing structures and the reconstruction of the Secondary Dam has been presented elsewhere in a paper by El Turki et al (1985).

2. LOCAL CONDITIONS

2.1. General

Wadi Qattarah catchment upstream of the dams is located East of Benghazi on the escarpment zone, at elevations between 150 and 650 ; areas are 1,224 km^2 and 1,285 km^2 at Main Dam and Secondary respectively.

Climate is typical semi-arid zone but the nearness of the Mediterranean assures an average yearly rainfall of some 230 mm from October to March mainly.

The geological set-up is a rather regular Miocene series featuring near-horizontal beds of dolomitic limestones alternating with some more chalky and marly strata. Dissolution certainly exists in the area but probably not at a large scale as was evidenced by the various investigation boreholes which encountered voids only occasionally. Infiltration is however highly developed through the discontinuity system of horizontal bedding connected by subvertical fracturing. Rock mass

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permeability is moderate (around 10^{-5} m/s). The aquifer is some 150 m and 120 m below ground level at Main Dam and at Secondary Dam site respectively.

Infiltration data are largely scattered depending upon the years ; for low water levels a progressive decrease is evident with time but for high water levels data and direct observations are too fragmentary to draw conclusions.

2.2. Main features of both Dams

Both dams are homogeneous embankments with similar design characteristics and similar appurtenant works. Fig. 3 and 4 show the plan view of both dams.

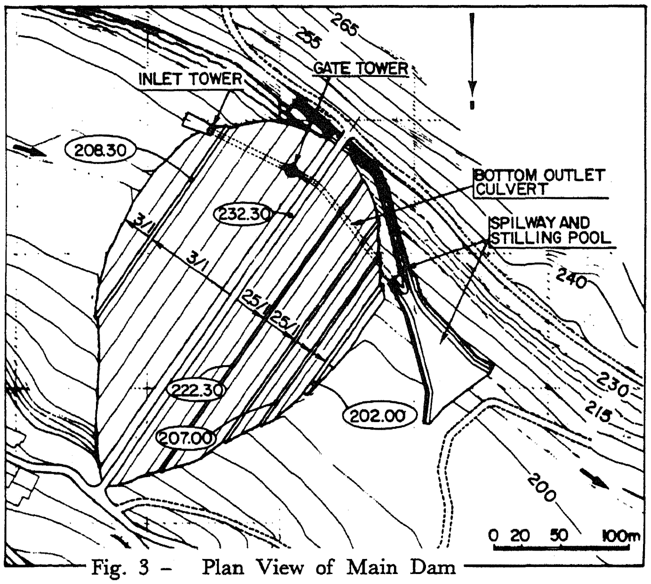

Fig. 3 – Plan View of Main Dam

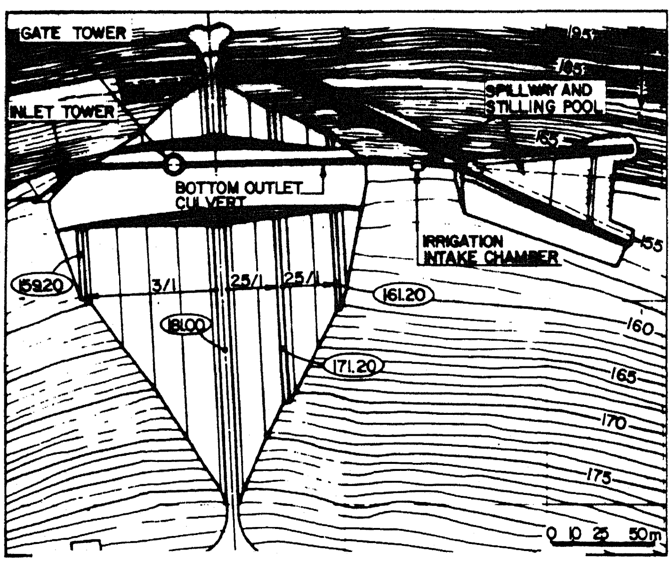

Fig. 4 – Plan View of Secondary Dam after Failure

2.2.1. Description of Main Dam Embankment

The maximum height of the embankment is 33.0 m and its crest length is 305 m. Fig. 5 shows the typical cross section of the embankment. The upstream slope of the embankment is 3.0 horizontal for 1.0 vertical (3:1). A 3-metre wide berm is located at El. 208.00 ; it was the crest of the upsteram cofferdam. With this berm, the mean upstream slope becomes 3.1 horizontal for 1.0 vertical (3.1:1). The mean downstream slope of the embankment is 2.7 horizontal for 1.0 vertical (2.7:1) including the three 2-m wide berms.

This embankment is an homogenous earthfill whose design includes an inside drainage network in the downstream portion of the embankment :

- A chimney drain including successively from upstream to downstream a fine filter layer 0.40 m thick, a coarse filter layer 0.40 m thick and a drain layer.

- This chimney drain is connected at its base with a horizontal drainage system generally resting on the foundation under the downstream portion of the fill. This horizontal drainage system includes a continuous layer of the fine filter material, 0.40 m thick, and six finger drains approximately 30 m apart, forming the horizontal outlets from the chimney drain. The core of these finger drains (area 1.40 m^2 each) is made of the drain material wrapped in the coarse filter material 0.40 m thick without any fine filter between the coarse filter and the earthfill.

- The horizontal drainage system is connected downstream to a rock-fill drain founded at variable levels below the foundation level of the embankment. A fine filter layer 0.40 m thick has been placed underneath this rock fill toe drain in contact with the foundation. This drain collects the flow from the horizontal drains and also from surface runoff.

A rough estimate of the maximum drainage capacity of this downstream drainage system indicates a value of the order of 5 l/s for a hydraulic gradient of 0.3 and permeability coefficients of 10^{-3} m/s and 10^{-4} m/s for the drain and fine filter respectively. The main limitation of the drainage capacity is due to the small area of the finger drains in the horizontal drainage system.

The embankment is partly founded on the limestone which outcrops on the left and right banks, representing 2/3rds of the area of the foundation surface, and partly on alluvial deposits in the lower part of the valley. This alluvial deposit rests on the limestone foundation some 17 m below at the lowest point. In order to provide a watertight link between the grout curtain in the limestone foundation and the embankment, a key trench filled with silty clay material has been excavated in the alluvial deposit. The key trench, 12 m average width at its base, has near-vertical walls on the upstream and downstream sides (4 in 1 slope). The longitudinal axis of the key trench coincides with the axis of the dam. A small narrow trench has been dug along the axis of the key trench in the limestone formation in order to install the concrete cut-off wall on top of the grout curtain. Like the key trench, this narrow trench has almost vertical sides (4 in 1 slope). The top of the cut-off wall, which is 4 m high and 1.50 m wide, is located generally a few metres underneath the rock line level.

Concerning the preparation of the rock foundation before placing the earthfill, it must be pointed out that no mention of preparation of the surface by sealing fissures and holes with grout or cement mortar has been found in the construction records. The Technical Specifications in the Tender Documents require treatment of the fissures and holes in the bedrock foundation undernearth the earthfill by filling them with a clayey backfill.

The condition of placing earthfill around the bottom outlet culvert has also to be mentioned. Both faces of the excavated trench in which the conduit lies are only 1 m from the concrete, which does not allow proper compaction of the backfill even by small hand-operated machines. This applies to the downstream portion of Main Dam culvert (Fig. 6).

In addition no particular drainage was provided around the downstream portion of the culvert. The filter blanket, part of the drainage system, appears to be located 1 m or 2 m above

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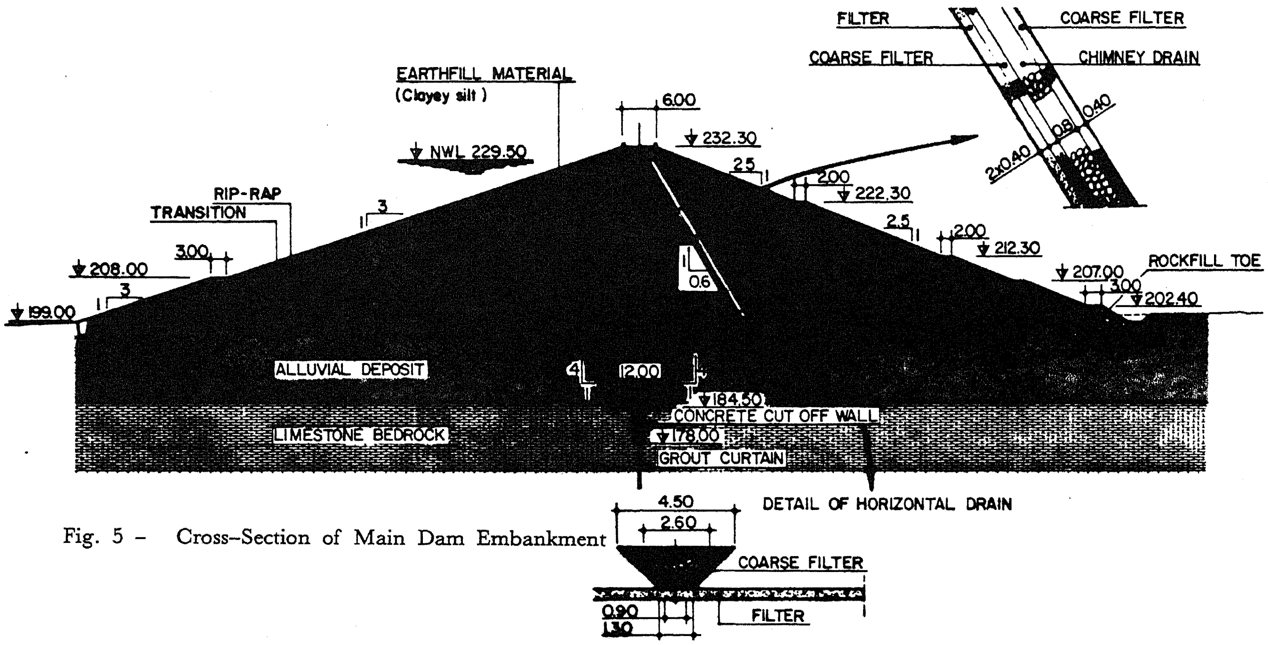

Fig. 5 – Cross-Section of Main Dam Embankment

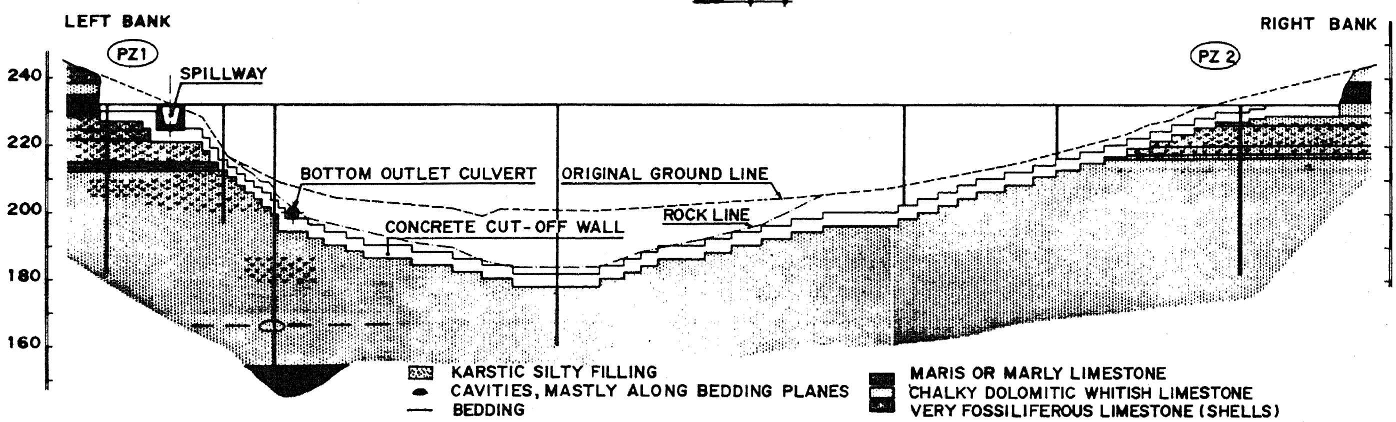

Fig. 6 – Longitudinal Cross-Section of Main Dam Embankment

the outlet culvert as well as the rockfill toe drain, whose base is about 3 m above the culvert.

Another feature of the design of this outlet culvert must be mentioned. The upstream part of the culvert from the intake structure to the gate tower is founded on alluvium through concrete piers. Consolidation of alluvium between the piers may have induced differential settlement of the alluvium and hence possibly allowed a seepage path to develop undernearth the culvert.

2.2.2 Description of Secondary Dam Embankment

The design of Secondary Dam is very similar to that of Main Dam. The crest length of the embankment is about 217 m and the maximum height is 28.6 m. Fig. 7 shows the typical cross section of the embankment.

The upstream and downstream slopes are the same as at Main Dam. Secondary Dam was constructed simlateneously with Main Dam. In this case too, the embankment is an homogeneous earthfill whose design includes features identical to those described previously for Main Dam, except that in this case the alluvial deposit has a maximum depth of 9 m.

2.3. Geotechnical Properties of Embankment Materials

With regard to the assessment of the safety of Main Dam and the causes of the failure of Secondary Dam, the significant embankment materials are the earthfill, filter and drain materials. The construction records indicate that both embankments were built with materials taken from the same borrow areas, and therefore no distinction will be made for their presentation below.

2.3.1. Description and identification material

The borrow area for earthfill is located about 2 km downstream of Main Dam site. The material was excavated from alluvial deposits. It is a clayey silt (classified CL2 in the USCS classification system) which has been identified as follows :

- The percentage of particle sizes smaller than 0.1 mm ranges from 75% to 95%.
- The percentage of particle sizes smaller than 2 microns ranges from 8% to 30%.
- The mean value of the specific gravity is 2.66 with a standard deviation of 0.03 (20 samples).
- The liquid limit WL ranges from 29% to 38%. The mean value is 33% with a standard deviation of 3% (80 samples).
- The plastic index PI ranges from 10% to 19%. The mean value is 14% with a standard deviation of 2% (80 samples).
- The shrinkage limit SL ranges from 10% to 14.5%. The mean value is 12%.

The natural water content generally increases with depth in the borrow areas. The average values ranges from 6% at 1 m to

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12% at 5 m depth. Almost all the material was excavated from a depth less than 6 m.

According to the standard Proctor compaction tests (compaction energy 600 kJ/m^3), the optimum moisture content Wopt ranges from 15% to 21% (mean value 18.1% with 1% standard deviation for 60 samples) and the maximum dry density Dd ranges from 16.7 to 17.9 kN/m^3 (mean value 17.3 kN/m^3 with 0.3 kN/m^3 standard deviation for 60 samples).

Mineralogical and chemical analyses were performed on 5 samples of the embankment material from Main Dam. The minerals were predominantly clayey with 20% to 30% kaolinite, 25% to 35% illite, 25% to 35% illite–smectite, 5% to 15% smectite and some (less than 5%) attapulgite. The chemical analyses show predominantly Fe_2O_3 and Al_2O_3 (50°/•• and 110°/•• respectively), some (30°/••) $CaCO_3$ and traces of monoxides and dioxides (CaO, MgO, NA_2O, K_2O).

Eight pin hole tests were performed on samples compacted at dry density and water content close to the optimum values from the standard Proctor tests. All show non dispersive properties for this material (class ND1). Nevertheless, the authors visited the borrow areas and observed, in some locations, typical erosion patterns (see Fig. 8) as if the material was dispersive. Khan (1983) reported results of chemical analysis carried out on 25 samples which indicated a high Na^+/Ca^{++} ratio. All 25 samples fall in zone C indicating the soil to be moderately dispersive according to Sherard et al (1976). The crumb test also gave similar results (Grade III), but no critical shear stress test as recommended by Arulanandan and Perry (1983) was made.

The clayey silt material has been placed dry generally. From the construction records of Main Dam, it was reported that the water content of the compacted soil ranged from 13.6% to 21.1% with a mean value of 16.4%, which is almost 2% less than the main value of the optimum water content from the standard Proctor compaction tests. The dry densities of the compacted soil ranged from 17.1 to 18.7 kN/m^3 with a mean value of 17.9 kN/m^3, which is 103% of the mean maximum dry density from the standard Proctor tests.

Regarding Secondary Dam, the water content of the compacted soil ranged from 15.2% to 17.6% with a mean value of 16.3% and the dry density ranged between 17.1 and 19.0 kN/m^3 with a mean value of 17.9 kN/m^3.

2.3.2. Description of filter and drain material

The filter and drain materials were made by crushing limestone. This limestone presents rather good engineering properties (low porosity and high compressive strength). The main features of the grain size curves of the fine filter material are the following :

- Dmax ranging from 6 to 10 mm
- D85 ranging from 3 to 7 mm
- D50 ranging from 0.5 to 1.5 mm
- D15 ranging from 0.15 to 0.5 mm

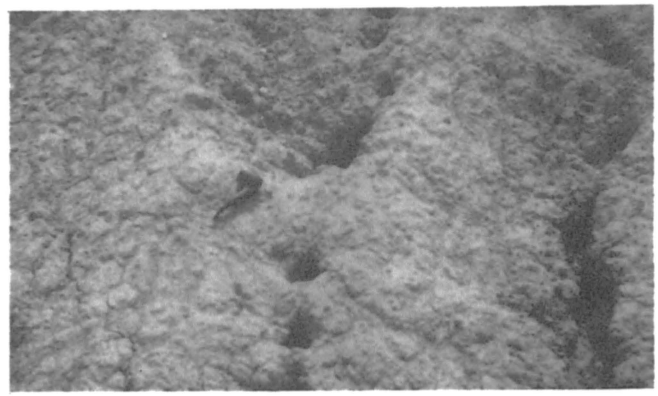

Fig. 8 – Erosion patterns observed in the borrow material areas

- Percentage of fines (below 80 microns) : generally 0%
- Uniformity coefficient D60/D10 : values ranging from 6.0 to 15.0.

The main features of the grain size curves of the coarse Lfilter and drain materials (apparently no distinction is mentioned between these two materials in the construction records) are the following :

- Dmax ranging from 50 to 150 mm
- D50 ranging from 12 to 50 mm
- D15 ranging from 8 to 20 mm (generally ranging from 10 to 15 mm)
- Percentage of fines (below 80 microns) : 0%

These results show that the coarse filter has a very good permeability and satisfies the current filter criterion (D15/d85 ≦ 5) with respect to the fine filter. The fine filter also satisfies the filter criterion (D15 ‹ 0.5 mm) as recommanded by Sherard et al (1984) since the d85 of the base material is 0.06 mm. According to Sherard et al (1984), such a fine filter is also acceptable for a dispersive clay.

2.4. Geotechnical properties of foundation materials

Underneath the alluvial deposits, the bedrock formation includes several kinds of more or less marly limestones. In the immediate vicinity underneath both embankments, the limestone has been termed as a dolomitic soft desintegrated porous limestone. Some unconfined compressive strength tests have been carried out on rock samples taken from boreholes in 1978 and 1979 through the main embankment. The results show a considerable scatter, with values ranging from 0.28 to 45.2 MPa for samples taken at similar elevations. This scatter may be due to the clay content (or marl content) of the samples. For samples exhibiting the lower values (0.28 to 1.14 MPa), undrained deformation modulus, if measured, would be of the same order of magnitude than that of the stiffer portions of the earthfill above. As the total vertical stresses appplied to the limestone by this earthfill reach 0.9 to 1.0 MPa, the top portion of the foundation certainly behaves as a compressible medium.

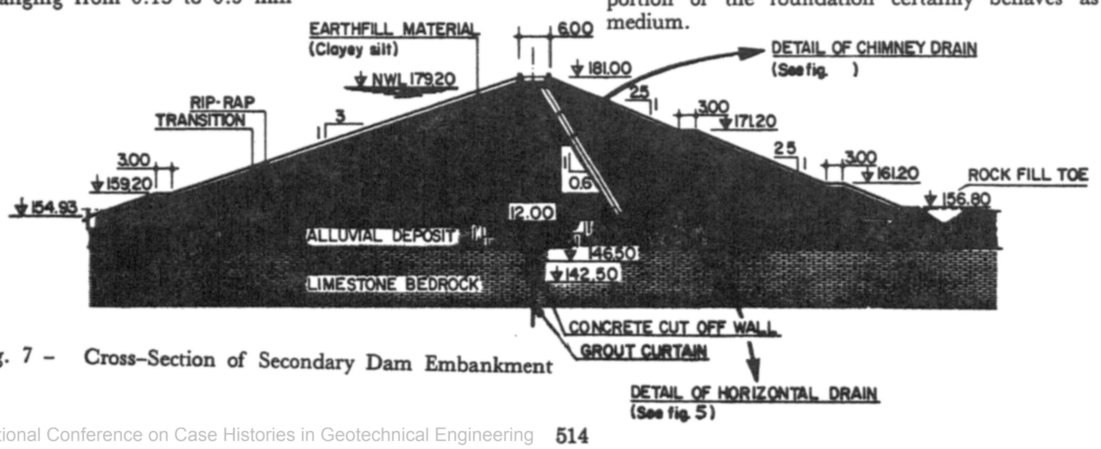

Fig. 7 – Cross–Section of Secondary Dam Embankment

Regarding the alluvial deposit, this material is described as clayey alluvium (silty clays, clayey silts, including gravelly layers and some blocks of weathered limestone). The field records mention that total loss of water was observed when drilling through the deposit. Due to the similar deposition conditions at Main and Secondary dam sites and borrow arreas, the geotechnical properties of the alluvium deposit are very likely to be the same in the three zones. The identification characteristics and mechanical properties of the clayey silt from borrow areas have been reported in previous sections. The only additional information of interest is the in situ unit weight and water content of this material. These data obtained for superficial samples are the following :

- dry unit weight of undisturbed soil samples ranged from 14.4 to 16.5 kN/m^3 with a mean value of 15.3 kN/m^3 (38 samples),
- natural water content ranged from 7 to 12% with a mean value of 9.6%.

Unfortunately, data from deeper samples are not available. The above results indicate that the superficial part of the clayey silt deposit is in a dry and relatively loose state with respect to the optimum conditions of the Standard Proctor Compaction Test.

2.5. Observational Data on Main Dam Embankment

The observational data concern the following parameters

- water level in reservoir,
- water levels in piezometers,
- settlements of embankment layers using cross arms monitoring system,
- total stress in embankment using total stress cells (during construction only)
- movements of crest and slopes of embankment using topographic monuments (or bench marks).

2.5.1. Water level in piezometers

The provisions for piezometric measurements consist essentially of :

- a row of nine vertical standpipes, numbered PI through PIX with their tops opening on the downstream berm at El. 207 and pressure inlet in the filter blanket,
- two vertical standpipes in the bottom part of the valley, P2 and P6, 5 and 100 metres from the downstream toe of the dam, respectively. The pressure inlet of P5 seems to be set in the alluvial clayey deposit, and that of P2 in the underlying limestone,
- an isolated standpipe, PZ2, at the right bank end of the dam axis, with top opening at El. 232.30 and pressure inlet at El. 182.30 in the dolomitic limestone bedrock.

Piezometric levels in these standpipes have been recorded almost continuously since December 1977.

Some of the piezometers stay constantly dry (or with a few centimetres of water at the bottom) even with high water levels in the reservoir. At all other standpipes (PIII, PIV, PV, PVI, PVII, PVIII, PIX, P2 and PZ2, a rise in the reservoir level produced a significant rise in the piezometer level as follows :

- changes in the reservoir level below El. 211.00 do not produce any change in piezometric levels.

- when the water level in the reservoir rises from El. 211.00 to 216.00, piezometric level increases are between a few centimetres and about 20 cm.

- from El. 216.00 to 219.00 the reservoir fluctuation produces a significant change in piezometric levels, between 15 and 55 cm at sandpipes located on the downstream berm and up to 200 cm at PZ2 on the right bank.

- the changes in piezometric levels occur soon after the corresponding changes in the reservoir elevation, as the time lag appears generally less than one day. This quick response is not at all usual ; the compacted clay should normally feature a coefficient of isotropic permeability K not higher than 10^{-7} m/s, with which time lags must be considerably longer.

The abnormally fast response of some piezometers prompts the fear that seepage has worked preferential paths. These paths may be entirely contained in the limestone bedrock and thus by-pass the grout curtain laterally. However, the typical cross-section suggests that more direct paths may have also developed after hydrofracturation of the thin fill in the narrow trench immediately above the concrete cut-off.

2.5.2. Total stresses in embankment

Thirteen total stress cells were also installed in the embankment during construction. One of these total stress cells (No. 10) is located in the fill just above the concrete cut-off (El. 183.70). Another cell (No. 9) is located at El. 199 on the top of the backfill of the key trench, just above Cell No. 10. Readings of total stresses during construction show that :

- for almost all cells, except Nos. 9 and 10, the total stress measured is equal or slightly larger than the computed vertical stress (using the actual bulk density of the earthfill).

- cells Nos. 9 and 10 show a very different trend. The measured stress is 65% (cell No. 10) to 71% (No. 9) of the computed total vertical stress.

2.5.3. Settlements of embankment monitored with cross-arms

Five cross-arms vertical elements have been installed. Three of them are located on dam crest (CSI, II and III) and the other two on the upstream and downstream slopes (CSV and CSIV respectively). They are of the USBR type with steel pipes 2 and 1.5 inches in diameter, spacing between crosses nearly 3 m. The installation of the cross-arm elements progressed along with the placing of the fill and measurements of vertical deformations were taken during the construction.

The vertical deformation curves at the end of the construction are roughly parabolic with the maximum settlement at mid-height (including the key trench backfill). This is the usual settlement distribution in an earth dam. It is worthwile mentioned that the settlement at CSII at end of construction and at the junction with the concrete cut-off wall is more than 3 cm which confirms the compressibility of the bedrock. From the distribution and magnitude of the settlements observed 8 and 9 years after the completion of the dam, the following remarks can be drawn :

- at the end of the 9-year period, the magnitude of the total post-construction settlement of the backfill in the key trench is 14 cm at CSI, 22 cm at CSII, 9.8 cm at CSIII and a few centimetres only in the alluvial deposit (according to the results of CSIV and V).

- the magnitude of the total post-construction settlement for all levels at CSV is about twice that at CSIV. The increase in soil saturation and the water load on the upstream portion of the dam may well explain the difference.

- the total settlement observed between February 1980 and February 1981 (1 year) represents an important fraction of the total settlement observed since the end of construction (9 years before) : 16% for CSI, 11% for CSII, 16% for CSIII, 14% for CSIV and 9,5% for CSV.

Thus, considerable settlement of the embankment is still developping, 9 years after the completion of the project.

2.5.3. Horizontal displacements of crest and slopes of the embankment

From the horizontal displacements of the crest and slopes taken from the bench mark measurements performed in 1980. It can be seen that the upstream slope has generally moved upstreamwards, with maximum displacements of 132 mm and the downstream slope downstreamwards with a maximum displacement of 70 mm.

The displacements of the crest are more erratic ; the right and central stretches have moved downstreamwards and towards the left bank ; the left stretch towards the right bank with an upstream component at Bench Marks No. 1 and 3, and, surprisingly enough, a large downstream component (50 mm) at Bench Mark No. 2.

The important displacement towards the right bank of the downstream slope near the left abutment (Bench Marks No. 1, 11 and 21) must be underscored. In places, the corresponding extension strain with reference to the nearest fixed point on the rocky edge may well exceed 0.5%. The development of significant extension strains in the area is confirmed by visual observation of the concrete parapet of the crest, between the spillway and the gate tower. The individual elements, 8 m long each making up this parapet, have separated or broken ; present openings are as follows : 25 mm between spillway right hand side wall and Element No. 1, 10 mm at a crack near the middle of Element No. 1, 12 mm between Elements No. 1 and 2 and 4 mm between Elements Nos. 2 and 3.

The above displacements reflect high extension strains (0.2 to 0.3%). Thus the presence of an extension zone in the earthfill near the contact with the steepest part of the left abutment (above El. 215.00) can be expected.

3. CHRONOLOGY OF SECONDARY DAM FAILURE

The following is a summary of the information gathered by the authors from the main witness, a technician in charge of the operation and maintenance of the electrical and mechanical equipment of the Wadi Qattarah Dams. According to the operation data, Secondary Dam had never impounded water until November 1977. Between December 13 to December 21, the reservoir level raised as follows :

Date	Reservoir Level
Dec. 13	164.65
Dec. 18	170.67
Dec. 19, 6.00hrs	176.00
Dec. 20, 7.00 hrs	175.62
Dec. 20, 9.30 hrs	175.48
Dec. 21, 9.30 hrs	175.24 (rain, check of the gates as usual)

Upon leaving the gate tower, on December 21, the technician went down the embankment slope along the spillway chute and over the outlet of the bottom conduit, on his way to the car, noting that no water was flowing out of the said conduit, an indication that the fixed-wheel gates had been properly closed. It was 10 a.m. approximately when he left the Secondary Dam area. At around 12 noon, a staff member informed the technician that he had just noticed muddy water flooding the toe of Secondary Dam above the bottom outlet (see Fig. 9). This observation had been made at 11.30 a.m., whereas the place was seen dry half an hour earlier.

Shortly after, the technician was back at Secondary Dam. A large discharge of water was flowing above the conduit in the area between the toe of the fill and the 1-meter diameter valve room. Soon the downstream slope started eroding, with blocks from the surface rip-rap ravelling down. Water under pressure spat from a hole in the slope above the conduit. Then, the retrogressive erosion ate up progressively the downstream half of the embankment (see Fig. 10).

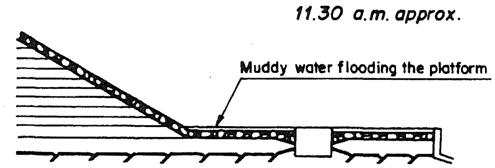

Fig. 9 – Sequence of Failure at Secondary Dam (11.30 a.m)

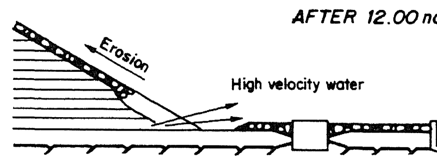

Fig.10 – Sequence of Failure at Secondary Dam (after 12.00 noon)

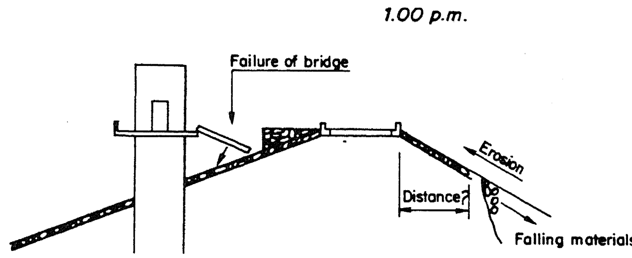

Fig.11 – Sequence of failure at Secondary Dam (1.00 p.m.)

At about 1 p.m., the service bridge between the crest of the dam and the gate tower fell down (see Fig. 11) and ten minutes later the retrogressive erosion broke through this crest, opening a breach to the reservoir; It took until 17 hours for the complete emptying, while the breach was widening and deepening; at no time, vortices or eddies were observed on the water surface in the reservoir.

4. MAIN WEAKNESSES OF THE DAMS AND POSSIBLE CAUSES OF FAILURE OF SECONDARY DAMS

The sequence of events in failure of the secondary dams as described above is typical of the phenomenom of piping.

As Post and Guerber (1973), Sherard (1986), and others stress, internal erosion results from dangerous seepage of water of diverse origins :

i) cracking due to differential settlement, as was the case at the Stockton Creek dams in California in 1950, and at Wister Dam in Oklhahoma in 1949 ;

ii) leakage either through the foundation/core contact plane, as at Hills Creek in 1970, or through one of the abutments, as at Fontenelle in 1965 ;

iii) horizontal cracking due to load transfer and hydraulic fracturing of slightly compressed zones, as at Hyttejuvet in Norway in 1966, at Balderhead in England in 1967, horizontal cracking causing wet seams in the cores of Manicouagan 3 in Canada, Yard's Creek in New Jersey, and El Guapo in Venezuela.

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iv) differential settlement near a fault or near an especially easily eroded, compressible foundation such as that at Baldwin Hills dam in the United States (1963) ;

v) low internal stress due to drying out and shrinkage during and after construction, especially in low dams in arid climates such as La Escondida dam in 1972.

The characteristics of the two Wadi Qattarah dams and the instrumentation results outlined above give reason to think that at least 4 of the above-mentioned causes of seepage (i, ii, iii and v) could well have been present : cracking due to differential settlement, as shown by cross-arm measurement and monitoring of stretching at the crest and face on the left-bank side ; leakage along preferential passages in the foundation, as demonstrated by the rapid response of piezometers to reservoir level rise ; horizontal cracking due to load transfer in the key trench fill, as seen from total pressure cell redings, as well as in the fill between the concrete culvert and the almost vertical sides of the trench through the rock ; and low internal stress due to drying out and shrinkage of the slightly to moderately plastic clay fill placed on the dry side of the Proctor optimum in a semi-arid climatic zone. To this can be added the drainage system which, although comprising filters and drains of satisfactory grain sizes, is very inadequate and badly situated.

As to how piping occured at the Secondary Dam, several mechanisms are conveivable, but so far none of then can be retained definitely as the one responsible for failure. Two of the mechanisms seem almost equally probable :

1) Piping developed entirely through the compacted fill around the bottom outlet culvert and directly from the reservoir. This is supported by the conviction that the backfill around the culvert was poorly compacted and the arching could have developped, thus leading to hydraulic fracturing of the fill.

2) Piping developed through the downstream portion of the compacted fill around the bottom outlet culvert. The mechanism would be the same as (1) above but with the water head triggering the hydraulic fracturing initially applied at a point on the rock-fill interface intermediate between the cut-off wall and the downstream toe. The most conceivable path for bringing the full reservoir head to the said point is a solution channel in the limestone foundation.

In the opinion of the authors the second mechanism is the most likely.

5. CONCLUSIONS

Examination of the sequence of events leading to failure of the Secondary Dam would indicate that failure was brought about by internal regressive erosion of the clay fill, starting at above the bottom outlet culvert.

Examination of the characteritics of the Main and Secondary Dams, together with Main Dam monitoring results, would indicate that there are several possible causes of water infiltration :

i) cracking of fill due to excessive differential settlement ;

ii) cracking of fill (or low internal stress) due to excessive drying out and shrinkage ;

iv) water flow along preferential passages in the limestone foundation (solutions ducts).

From examination of the possible causes of Secondary Dam failure, it can be thought that the most probable mechanism for internal erosion resulted from a combination of flow in foundation solution ducts and hydraulic fracturing in fill around the downstream end of the bottom outlet culvert. This explanation does not exclude the involvement of other factors (cracking due to differential settlement or shrinkage) in facilitating the triggering of the mechanism.

Furthermore, the lack of adequately dimensioned filters and drains in the right places means that the phenomenom, once started, could not be efficiently combatted to prevent failure.

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