Marmarik Dam Investigations and Remedial Works

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SYNOPSIS. Marmarik dam is a multipurpose embankment dam on the Sevan – Hrazdan cascade in Armenia. The dam is situated in one of the most seismically active regions in Armenia and in the vicinity of the reservoir numerous landslides could be seen. The dam was commissioned in January 1975 and twenty days later significant subsidence of the clay core occurred causing 14m settlement of the dam crest. The dam has never been rehabilitated and the reservoir has never been impounded.

The dam was investigated by JacobsGIBB ltd as part of the World Bank funded 'Technical Investigation of 60 Dams'. In addition to the slope failure, further issues include high regional seismicity, landslides adjacent to the dam and reservoir, inadequate spillway capacity, rehabilitation of the derelict outlet works and concerns regarding the foundation cut-off.

INTRODUCTION

Marmarik dam, situated in Kotayk Marz in Armenia, was originally designed to provide water for the future aluminium mining industry, a cement factory, two thermal power plants, irrigation of 2,000ha and flood water regulation. However, as the aluminium mining industry was never developed the dam changed ownership and the new owner became the Ministry of Water Resources. Marmarik dam is a part of the Sevan – Hrazdan cascade which significantly contributes in overall regional energy balance and provides water for irrigation systems and six power plants.

The dam was commissioned in January 1975 and twenty days after the commissioning significant instability of the embankment occurred, causing a 14m settlement of the dam crest over half of the dam crest length. Immediately after the subsidence a local company was commissioned in 1975 to investigate causes of the dam failure. It was found that the failure occurred as a result of high pore pressure in the clay core that was placed with a high moisture content. The dam has never been rehabilitated and therefore the reservoir has never been impounded. The river is diverted in a

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tunnel through the left abutment. This uncontrolled diversion has been left in operation since the construction period.

DESCRIPTION OF THE DAM

Embankment

Marmarik dam, Figure 1, was originally designed as a 64m high embankment with a clay core and compacted fill shoulders. The original dam crest was at 1914masl. The design cross section is shown in Figure 2. The shoulders were originally designed to be built of gravel from a borrow area some 5km downstream of the dam. However, only the bottom 5m of the embankment was constructed from gravel as the further use of the gravel borrow area was not permitted. Thereafter the embankment shoulders were constructed from a compacted sandy silt from borrow areas within the reservoir, but to the original design slopes and with no filters.

The central part of the dam is founded mainly on granular river alluvium and the abutments are founded on a thick layer of cohesive colluvial deposits. The designed foundation anti – seepage measures comprise a cut – off bored secant pile wall constructed up to 30m deep through the central part of the alluvial foundation and a grout curtain through the colluvial foundation at the abutments.

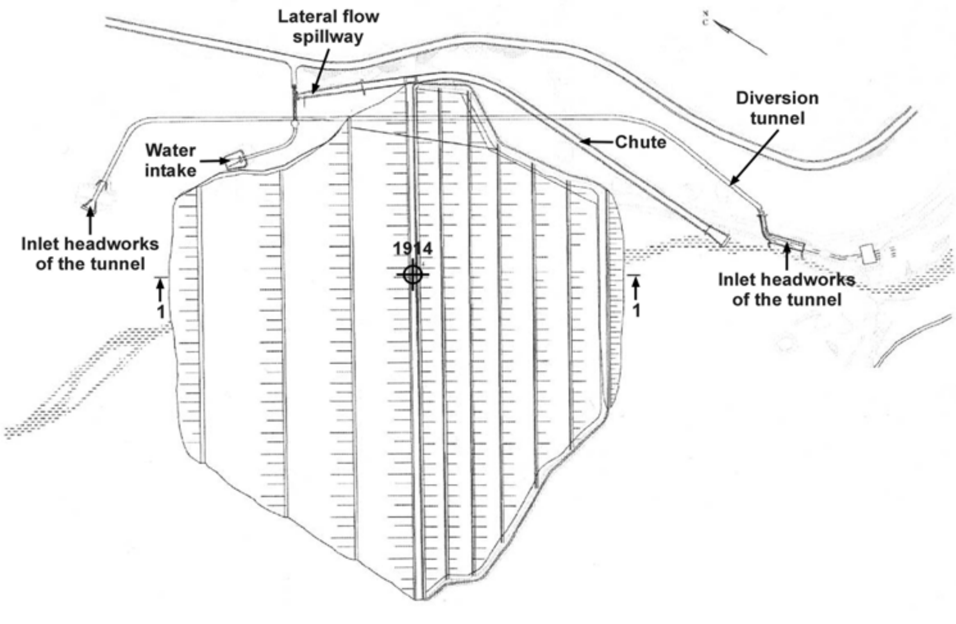

Figure 1. Marmarik reservoir- plan

Diversion tunnel

At its present state the river is diverted into a 3.2m diameter, D shaped diversion tunnel designed for a temporary condition, for a flood of $50m^3/s$ with a return period of 20 years. Since its completion, there have been three occasions on which the incoming flood exceeded the designed value but the flood was absorbed in the reservoir storage volume without risk of overtopping the dam. There is a side weir at the outlet end, which permanently maintains a minimum water level of 1.4-1.5m in the lower section of the tunnel. For that reason the complete inspection of the tunnel has never been carried out in the past.

Spillway

The spillway, situated on the left abutment comprises:

- A 60m long side-channel inlet weir
- A culvert under the dam crest
- A 6m wide discharge chute with a variable gradient

Outlet Works

The outlet works consist of a tunnel leading to a short connecting shaft to the diversion tunnel which is 11m below. The connecting shaft contains two 1.0m diameter outlet pipes which are cast into mass concrete which fills the shaft. A 50m deep, 6m diameter gate shaft is located at 5m offset from the connecting shaft. The gate shaft contains an emergency closure gate and a maintenance gate for each pipe.

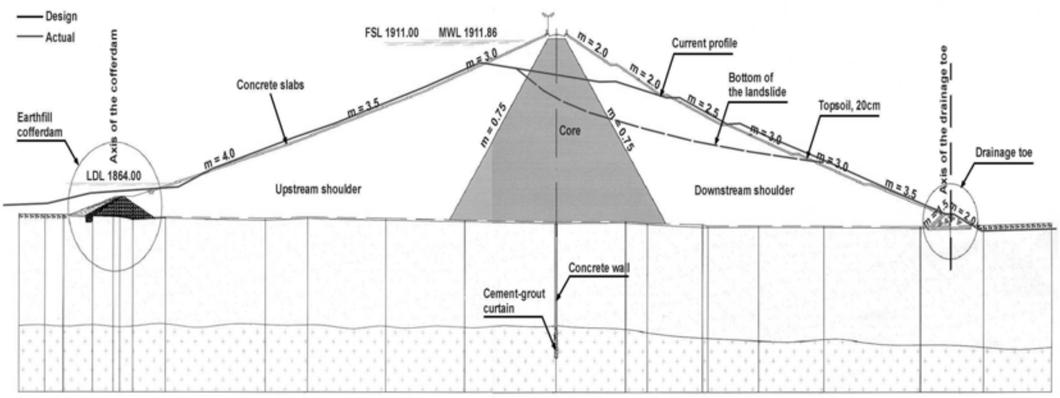

Figure 2. Cross Section 1-1

INVESTIGATIONS CARRIED OUT

Immediately after the subsidence of the dam, a local Armenian company was commissioned to investigate causes of the dam failure. The investigations were carried out between 1975 – 1978.

Under the 'Technical Investigation of 60 Dams' the following investigations and surveys were undertaken during 2002 - 2003:

- Topographic survey of the dam and the diversion tunnel
- supplementary ground investigations of the dam
- microseismic survey to establish site specific seismic parameters
- landslides hazard assessment and landslide ground investigation
- investigations of the diversion tunnel
- investigation of the efficiency of the foundation cut-off

The investigations undertaken during 2002-2003 are described below in more detail. Based on the results of the investigations and the findings of the investigations carried out during 1975-1978, geological, geomorphological and seismic conditions at the dam site were assessed as well as the status of the dam, foundation anti- seepage measures and the diversion tunnel.

Supplementary ground investigation

Supplementary ground investigation carried out to validate previous investigations included 480m of drilling through the dam, trial pitting, in – situ permeability testing and laboratory testing.

Microseismic survey

Microseismic survey comprised the following works:

- Seismic Refraction- carried out at 48 measuring points in the reservoir and 24 measuring points on the dam
- Measurements of ground micro - vibrations by using SMACH –SM and OMNILIGHT instruments - p-wave velocities were recorded in the surface deposits and in the bedrock, as well as the peak horizontal accelerations, vertical geomagnetic field and the distribution of predominant frequency spectra

Landslides hazard assessment and landslide ground investigation

Landslides of a seismogenic origin are widespread along the whole length of the southern (right) bank of the Marmarik River canyon. Four potentially hazardous seismogenic landslides were identified within the Marmarik reservoir area that may influence the dam safety, namely landslides N1 to N4. The landslides are shown in Figure 3. Landslide hazard assessment was carried out based on the analyses of satellite images and aerial photos that were taken in 1948, 1976 and 1986 as well as the field surveys carried out in 1975-78 and 2002.

Landslide N1 is located some 2 km to the S-SE of the Marmarik Dam, in the upper reaches of the Kiarkhana River, which is a lateral inflow of the Marmarik River and as such it poses a low hazard to the dam. The landslide was not investigated further.
Landslide N2 is located some 250m to the south of the dam. The landslide is situated close to the confluence of Kiarkhana with the Marmarik river. During 1969-1974, the material from the toe of landslide was excavated for construction of the Marmarik dam. The excavation destabilised the landslide leading to a development of presently active secondary landslides. The landslide was investigated during 1975-1978 site investigation. Thickness of the landslide varies from 30 to 80m, total volume is about $94x10^6 m^3$.

Landslide N3 is 1.6km upstream of the dam and it was reactivated a number of times in the past, most recently during dam construction when the soil from the toe of landslide was excavated and used for the fill material. The landslide was investigated during the 1975-1978 site investigation. Thickness of the landslide varies from 40 to 60m, the total volume is about $16x10^6 m^3$.

Landslide N4 is 5.2km upstream of the dam and at its toe, it branches into two landslides separated by some 700m. This landslide is the most distant from the dam, but it is the largest in volume. If it is triggered it could dam the Marmarik River and create a lake which, if the natural dam is breached, could induce a flood inflow into the reservoir. This landslide was investigated during 2003. It was found that landslide comprises a layer of rock debris with a soil matrix up to 50m thick, over a thin slip surface that overlays the in - situ rock.

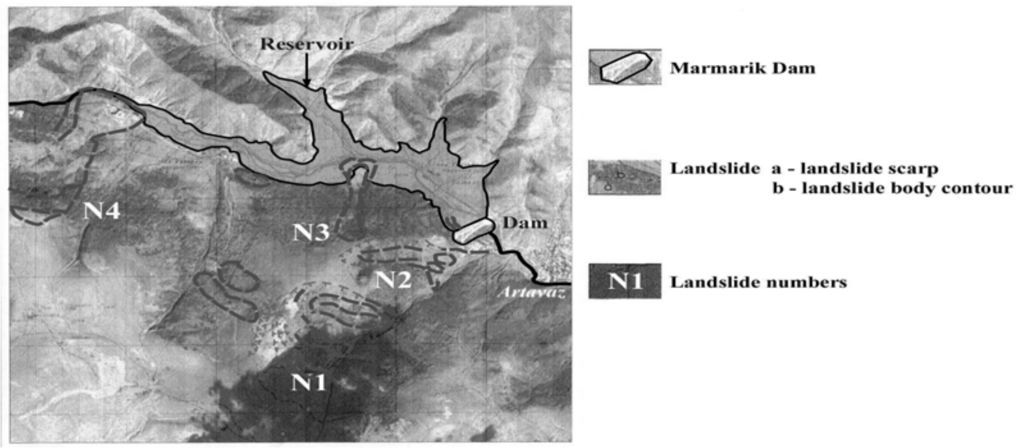

Figure 3. Landslide hazard map for the Marmarik reservoir

Investigations of the diversion tunnel

Investigation of the diversion tunnel comprised the following:

- Initial walk through and visual inspection
- Intrusive drilling through the tunnel lining
- Non destructive testing using a calibrated Schmidt hammer and a hand held ultrasonic meter to determine quality of the concrete.

Investigation of the efficiency of the foundation cut-off

As the reservoir has never been impounded, there is a significant uncertainty about the efficiency of the foundation anti-seepage measures, especially the ones through the alluvial foundation. The investigation of the effectiveness of the cut-off through the alluvial foundation was therefore accomplished by carrying out water pressure tests and indicator tests in the test section located in the centre of the river channel. The test section comprised three holes located 15m u/s of centreline (Hole1), 5m u/s of centreline (Hole 2) and 5m d/s of centreline and drilled down to the bedrock.

The water pressure tests were used to measure the difference in response of piezometers (placed in the foundation material upstream (Hole1) and downstream (Hole 3) of the cut – off) to water pressure applied in Hole2 drilled upstream of the cut – off. In Holes 1 and 3 piezometers were installed 5m below the fill/alluvium interface and 5m above the alluvium/bedrock interface. The difference of the response in Holes 1 and 3 is a measure of the permeability of the cut –off.

The water pressure test method was supplemented by the introduction of an indicator (salt solution) into the borehole and a comparison of the concentrations of the indicator throughout the borehole.

GEOLOGICAL CONDITIONS

Geology of the dam site comprises deeply weathered and fractured granodiorites and metamorphic complex of the Oligocene age. The dam site is located in fault-controlled river valley following the trend of a major northeast to southwest trending regional fault.

The central part of the embankment is founded on alluvial deposits (coarse sandy gravel) which fill the entire river valley and which are underlain by weathered granodiorite. The alluvial deposits vary in thickness between 10m and 30m and have a hydraulic conductivity of 10^{-4} to 10^{-5} m/s.

On the abutments the embankment is founded on colluvial materials, mostly silty clays. The colluvium covers the valley sides to a thickness of up to 20m and is derived from the weathering of granodiorites, with landslides in some areas. The hydraulic conductivity of the colluvium is 10^{-6} m/s.

Seismological conditions

Regional seismicity
Marmarik dam is located in a highly seismic area of Armenia. Some 13.7km north of the dam site runs the largest and most active Pambak – Sevan Fault. This is the main regional fault, 490km long, which in the past generated earthquakes of magnitudes up to 7.4. Also, very close to the site (5.2 km away), to the west of the dam, is the Garni fault, 198km long, which in the past generated earthquakes with magnitudes up to 7.0. The dam is directly situated on Marmarik fault, 30km long which joints the Garni fault. However, as no tectonic activity has been registered along the Marmarik fault in the Holocene, the fault is regarded to be seismically inactive.

Seismic design parameters
Seismic design parameters have been assessed based on the methodology given in Reference 1 as well as the site specific seismic hazard assessment.

The method in Reference 1gave the following design accelerations (return period of 475 year):
- Ground acceleration: a_{pk}= 0.144g
- Acceleration at the dam crest: a_{pk}= 0.555g

Site Specific Seismic Hazard Assessment was carried out using Deterministic Seismic Hazard Assessment (DSHA) and Probabilistic Seismic Hazard Assessment (PSHA). For site specific response the results of the microseismic survey were used.

The DSHA was used for assessing the maximum credible earthquake (MCE). Two past earthquakes were analysed; Mmax=7.5 along Pambak – Sevan fault and Mmax=7.1 along Garni fault. These earthquakes produced a peak horizontal acceleration of 0.44g and 0.82g at the dam's base and the crest respectively.

The PSHA produced the following peak horizontal acceleration:
a = 0.32g at the base and 0.6g at the crest (Return period of 100years)
a= 0.43g at the base and 0.81g at the crest (Return period of 250years-magnitude saturation occurs after 250years)

Based on the above analyses, the following design peak horizontal accelerations were recommended for checking stability of the dam:
- OBE= 0.32g at the base and 0.6g at the crest (Return period of 100years)
- MCE= 0.44g at the base and 0.82g at the crest (return period of large number of years)

Liquefaction analysis

Liquefaction assessment of the fill material was carried out based on the particle size distribution that did or did not liquefy during past earthquakes, Reference 2, and also the methodology given in Reference 3. According to the Japanese Seismic standard, the liquefaction potential is evaluated by calculating the liquefaction resistance factor, F_L. A soil layer having the liquefaction factor $F_L < 1.0$ is susceptible to liquefaction. An $F_L < 0.6$ was obtained for the fill in the top 10m of the dam (slipped mass) for an average seismic acceleration of 0.5g. Therefore some 60% of reduction in shear strength properties for that zone could be expected to occur during a strong earthquake.

STUDIES CARRIED OUT

Hydrological and Flood Routing

Two methods were used to analyse the flood inflows into the reservoir. The first, the SNIP method (Reference 4), is based on standard Russian techniques and is in general use in Armenia. The second is a statistical method that uses all annual maxima flow data recorded in the region and is derived from the approach developed during investigation of floods in the British Isles (Reference 5). The following results have been obtained for the 1:10,000 year peak flow:

- Regional Method: 147 m^3/s
- SNIP: 138 m^3/s

In addition to the 1:10,000 year flood, the flood that would result from breaching of the landslide dam due to reactivation of the Landslide N4 (see above) was also considered. The estimated peak inflow for this scenario was 1920m^3/s, with a volume of 2.4 million m^3.

The flood routing was carried out for the event of a 1:10,000 year flood as well as the event of a failure of a dam created by the N4 landslide. The flood routing was done for the existing condition (empty reservoir), for the design condition with the dam at its full height (FSL at 1911masl) and for an intermediate condition (partial impoundment).

Foundation seepage

Seepage through the dam foundation was analysed for two typical sections, namely for the deepest section with the piled cut-off and the abutment section with a grout curtain only. For a conservative assumption that the anti-seepage measures are ineffective, the total leakage through the dam foundation was assessed at about 100l/s.

Stability analyses

Existing condition of the embankment
Stability analysis of the upstream and downstream slope of the dam at its present condition was carried out using the parameters obtained from the investigations. The analyses demonstrated that the dam was stable with the reservoir empty. However, if impounded, the dam would be unsafe during rapid draw down (u/s slope) and steady seepage (d/s slope).

Stability after the remedial works to the embankment are implemented
Stability analyses was also carried out for three options for the remedial works. The embankment remedial works were developed so that minimum required factors of safety were satisfied for all loading conditions.

Stability of the landslides
The analyses carried out for the Landslide N2 showed that for sliding occurring along the predefined slip plane, factors of safety obtained were lower than unity even in the aseismic conditions. For possible new slip surfaces occurring within the landslide material, factors of safety obtained in aseismic conditions were higher than unity. However, in the case of an earthquake, slippage would occur. The slippage would most likely occur in a direction perpendicular to the ground contours, towards the Kiarkhana river and away from the dam and therefore would not directly affect the dam safety.

It was shown that the stability of the Landslide N3 is largely influenced by the reservoir water level. If the reservoir is filled the landslide would be re-triggered. The volume of the unstable mass was estimate to be 200,000m^3. It was shown that this mass would immediately raise the reservoir level by some 20cm. In addition a wave of 1.5m height would be induced. Such a wave, with its run up of some 2.8m would therefore need a minimum freeboard of 3m in order to prevent the dam from overtopping if the reservoir was full.

The volume of a potentially unstable mass for the Landslide N4 was assessed by stability calculations to be 2400m^3/m of the landslide length. That volume could create a 21m high natural dam which could impound a 2.4 million m^3 lake. As the river flow is some 3-4m^3/s the volume would be filled within a few days. In a major storm event this could take less than one day. The landslide ‘dam’ has been considered as an earth embankment and analysed for a dambreak. The analysis indicates a peak flood flow of 1920m^3/s and a flood volume of 2.4 10^6 m^3 (see above).

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

SUMMARY OF FINDINGS AND THE REMEDIAL WORKS

Embankment

Current crest elevation is approximately at 1900masl for a good part of the dam. Presently the upper part of the failed embankment material forms the top part of the core and the downstream shoulder. There is a very high perched water table within the dam body. Stability analyses demonstrate that the dam is stable in its present condition. However, if impounded to a level 3m below the current crest, the dam would be unsafe during rapid draw down (u/s slope) and steady state seepage (d/s slope). Furthermore, due to the high seismicity of the region, peak ground accelerations at the crest of about 0.6g could be generated. These accelerations are likely to cause liquefaction and strength reduction in the loose landslide material in the top part of the downstream slope and further contribute to the embankment's instability. It is therefore proposed to rehabilitate the embankment to improve its safety. Three options are developed as follows:

- Option 1 - Reinstate the dam to the full height with the crest at 1914 masl; Full storage level at 1911masl, total storage volume $36x10^6$ m^3
- Option 2 - Reinstate the dam to elevation of 1905masl; Full storage level at 1902masl, total storage volume $24x10^6$ m^3
- Option 3 - Reinstate the dam to elevation of 1889masl; Full storage level at 1886masl, total storage volume $10x10^6$ m^3

The earthworks proposed for the above options are shown on Figure 4.

Foundation anti - seepage measures

Foundation anti – seepage measures in the central part of the dam comprise a secant bored pile wall that was constructed through the granular alluvium into the bedrock. The field tests carried out in the deepest section indicated that the cut-off would reduce the overall foundation permeabily and the leakage would not exceed 100 l/s in the worst case scenario. Nevertheless, to reduce the uplift under the downstream shoulder, it is recommended that, for all three options of the embankment remedial works, 20 m deep, 200mm dia toe wells are installed along the downstream perimeter at 10m centres. The wells will comprise a perforated plastic tube wrapped in geotextile and placed inside a hole in a sand surround. Each well will discharge water into a collector trench which runs along the perimeter of the dam.

Landslide hazard

Four potentially hazardous landslides were identified in the vicinity of the dam. The landslides N1 poses a very low hazard to the dam. The landslide N2 is also likely to pose a low hazard, but because of its proximity to the dam it is recommended that monitoring instruments are installed in two monitoring profiles.

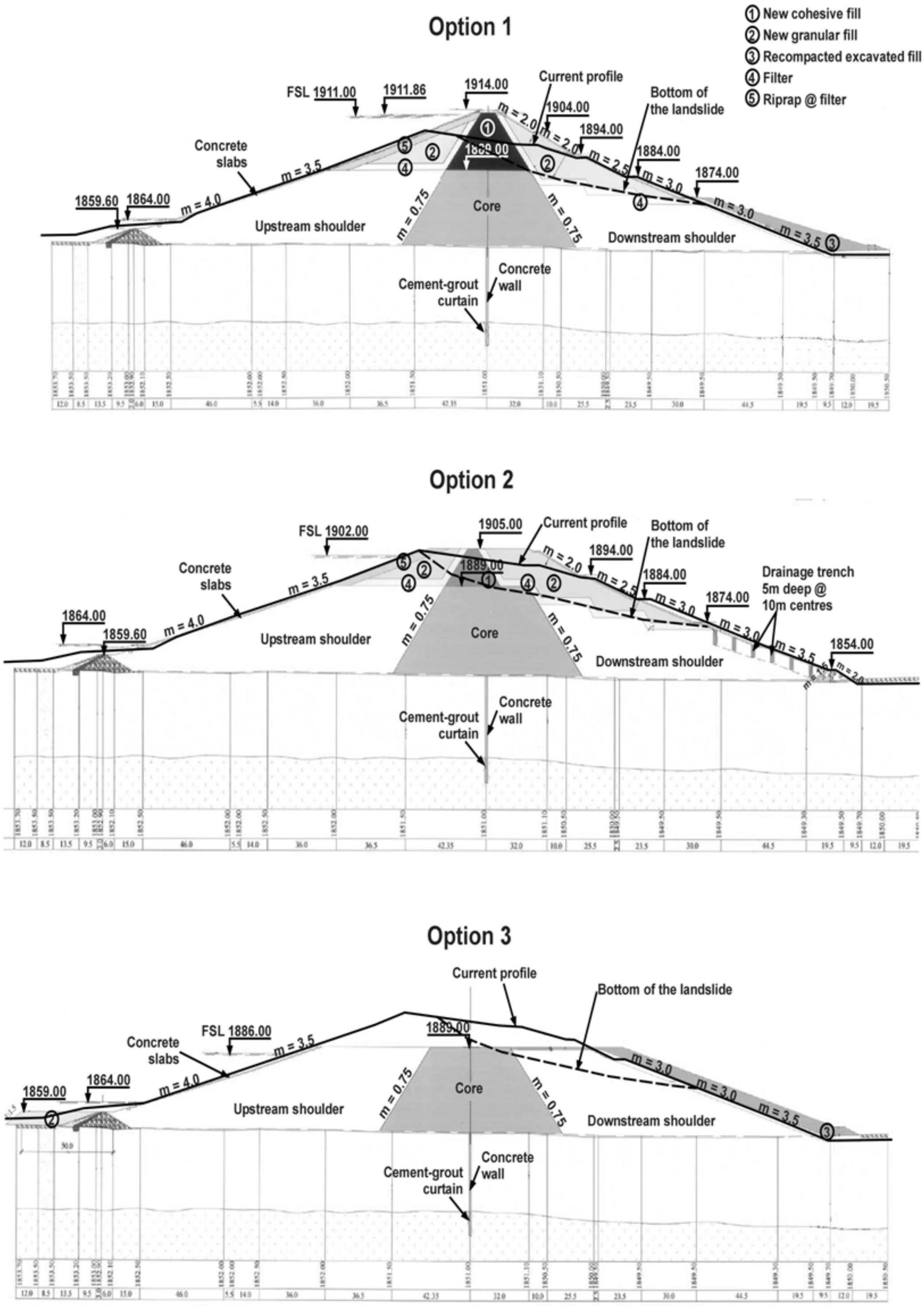

Figure 4. Options for dam rehabilitation

If the Landslide N3 slides into the reservoir it could create a 1.5m high wave and an allowance in the freeboard of 3m is made to accommodate the runup of such a wave. This landslide will also be monitored in two monitoring profiles.

If the landslide N4 collapses, it could block the Marmarik river and create a natural dam which if breached could create a 'dambreak' flood. It is recommended to install 1m high fuse gates (HydroPlus or similar) over the whole length of the spillway crest which could be activated should the landslide occur and the reservoir level needs to be lowered. Alternatively the spillway could remain conventional but the freeboard could be increased to 4m by provision of a 1m high concrete crest wall. It is also proposed to install monitoring instruments on the landslide and monitor the slope movements.

Diversion Tunnel

In its present state the river is diverted into a diversion tunnel designed for a temporary condition. The tunnel was inspected and investigated. The tunnel lining is of a satisfactory strength and the voids between the concrete and the rock are only of a limited extent. The tunnel is therefore considered to be stable in the short term. However the following remedial measures are recommended to enable its operation in the long term:

- Mass concrete plug upstream of the inlet pipes
- Consolidation grouting as a circumferential fan to a depth of 15m around tunnel over a 50m length downstream of the plug and backgrouting of tunnel lining in areas of voids.
- Replacement of tunnel invert downstream of plug
- Drainholes to be incorporated into invert to minimise hydrostatic loading.

Spillway

The existing spillway is in poor condition and requires substantial remedial works (Option 1). For Options 2 and 3 a new spillway is required at a lower level.

Outlet works

The outlet works require substantial refurbishment.

COST ESTIMATE FOR THE REHABILITATION OPTIONS

Costs for the three rehabilitation options are as follows:

- Option 1 - $10,5 M
- Option 2 - $7.5M
- Option 3 - $5.3M

A cost of decommissioning and breaching of the dam was estimated to be around $5M. It is likely that the client will go ahead with the rehabilitation Option 2.

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