Design of an Asphaltic Concrete Core Water Retaining Embankment in Iran

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ABSTRACT

A water retaining embankment was designed for the Miduk Copper Mine, near Shahr-e-Babak, in South-East Iran. This embankment forms part of a larger scheme for a down-valley thickened tailings storage facility which includes a tailings retaining embankment, decant system and associated emergency spillways. The water retaining embankment is located downstream of the tailings embankment. It is designed to collect bleed water and tailings surface and natural catchment rainfall runoff from the tailings storage facility. Water intercepted by this embankment is to be returned to the plant via an outlet pipe constructed through the embankment.

Selection of an asphaltic concrete core and the design of the water retaining embankment were based on two main criteria: the shortage of suitable core material and the associated costs of obtaining that material. This paper discusses the design process, including aspects of site selection, geotechnical investigation, material selection, stability analyses, seismicity, altitude and climate.

1 INTRODUCTION

The Miduk mine site is located 42 kilometres North-East of Shahr-e-Babak in Kerman Province, Iran. The general location is presented in **Figure 1** below.

Figure 1 - General Location in Iran

Australian Tailings Consultants (ATC) were contracted to undertake a feasibility study and then later design of the Tailings Storage Facility (TSF) for the Miduk Copper Project. Design of the TSF was completed in July 2003. Construction is currently underway.

2 PROJECT BACKGROUND

The Miduk Copper Project is a copper deposit which occurs within a fractured porphyritic granodioritic host that intruded into a sequence of early tertiary aged andesitic volcanic rocks. The total ore reserve is calculated to be 170 million tonnes with an average grade of 0.8% copper. The processing is expected to produce 145.5 Mt of tailings over an expected 30 year mine life.

The project is managed by Olang Mining Services Company on behalf of the owner National Iranian Copper Industries Co (NICICO). Production is scheduled to start in late 2004.

3 WATER BALANCE ISSUES

Water is a valuable resource in Iran. The minesite intends to make-up water from a nearby bore-field. The purpose of the water retaining embankment is to intercept water from the TSF including rainfall runoff from the natural catchment and tailings surface, groundwater seepage, bleed water and consolidation seepage (from the tailings). For the minesite, the greater the volume of water collected by the water retaining embankment, the lower the demand on pumping from the bore-field.

Water intercepted by the embankment will be pumped back to plant. Under normal operating conditions this dam will not store water. It is estimated that the plant water requirement will be greater than the expected inflow into the water retaining embankment **[Ref 1]**.

4 ALTITUDE AND CLIMATE

The Miduk minesite is located in a mountainous and arid region of Iran. Elevations around the site range from RL 2250 m up to RL 2842 m. Seasonal temperatures typically range from sub-zero in winter to the mid thirties in summer. The wet season is from November to April, with snow during the winter period. The average annual precipitation is estimated to be 250 mm.

5 SEISMICITY

Iran has a high seismic hazard risk. Three major faults exist in the Miduk region, Rafsanjan Fault to the east, Shahr-e-Babak Fault to the south, and the Anar Fault to the north. Seismicity data shows very few records of movement on these faults; this could be for several reasons including low level of activity on these faults compared with other parts of Iran, the relatively remote location, and poor seismograph coverage for this area (i.e. smaller earthquakes are not detected) **[Ref 2]**.

Because the geology of the Miduk area is very young compared to most of the Earth's surface (Tertiary age or younger), the attenuation of the seismic waves with distance is high.

Two earthquake types were analysed for the Miduk site – the Operating Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE). The return periods for these two earthquake types were selected as the 1,000 years and the 30,000 years respectively **[Ref 3]**. The Peak Ground Acceleration (PGA) calculated for each of these earthquakes was 0.35g and 0.8g respectively **[Ref 4]**.

6 SITE SELECTION

Site selection depended on several factors, these being the most efficient embankment alignment, the required storage capacity, and the results of the geotechnical investigation. The most efficient embankment alignment was selected to reduce material quantities and costs. The alignment also considered the suitability of the plinth alignment i.e. steepness of abutments. The embankment location was checked to see if the required peak event storage capacity was available.

7 GEOTECHNICAL INVESTIGATION

Investigations were undertaken in 2002 to characterize the site geology and available borrow material. These investigations included geological mapping of the proposed dam site, test pits in the proposed borrow areas, boreholes drilled in the proposed water retaining embankment site, and a review of the site geology.

The site investigation revealed bedrock consisting of andesite and andesite porphyry dykes, with surface deposits of alluvial and colluvial materials. A tectonised zone upstream of the right abutment and five minor faults were also noted during the site investigation. The faults encountered were analysed by an experienced geologist and an experienced seismologist. It was agreed by both that the faults in the vicinity of the dam were not active.

The alluvial and colluvial soils varied in depth from 0 m to up to 10 m. Typically the greatest depth of alluvials and colluvial soils were located in the gullies/drainage pathways.

The site investigation revealed groundwater flow in the order of 2 – 15 L/s through the alluvial soils (varies seasonally). Pressure packer testing showed the bedrock to have losses less than 2 Lugeons, however one borehole showed anomalous results of total loss. Those results were thought to be due to failure of the mechanical packer to seal the hole adequately.

8 EMBANKMENT SELECTION

Several embankment construction methods were assessed. The embankment types consisted of two main design types – a central core or an upstream membrane.

The central core options included:

- an impervious earth core, or
- an asphaltic concrete core.

The upstream membrane options included:

- a concrete deck,
- an asphaltic concrete membrane, or
- a geomembrane.

Concrete gravity dams were not considered due to the high seismic risk and presence of faults at the dam site.

The main advantage of a central core embankment over an upstream membrane is the reduced length of the core and grout cap/plinth and grout curtain, and subsequently reduced volumes of materials required. Other advantages included a simpler set out, ability to store water during construction, and the design feature of the core being buried and therefore not subject to damage or vandalism **[Ref 1]**.

No suitable source of clay was located onsite during the borrow investigation. Because of this, the design of an impervious earth core embankment would be more complex. Large quantities of alluvial gravels and sands were available with fines contents varying between 4% and 36 % **[Ref. 1]**. Earth core embankments can be constructed using moraine materials, but at Miduk the fine material exists in lenses within coarse alluvial material. A minimum fines content of 20 % would be required for this type of embankment, and obtaining a consistent quantity of fines was considered to be difficult.

8.1 Asphaltic Concrete Core Selection

More than 70 asphaltic concrete core (ACC) embankments have been constructed to date, with the highest the Storglomvatn Dam in Norway standing 125 m **[Ref. 5]**. Typical core widths range between 0.4 m and 1.2 m depending on the final height of the embankment **[Ref. 6]**. The core is placed above a reinforced concrete grout cap/plinth. Most ACC embankments less than 50 m in height are constructed with a vertical core which is supported laterally by a transition zone (grain size range of 0 to 200 mm), and then rockfill. The transition zone is not a filter; it merely provides lateral stability for the asphalt core during compaction, as well as providing drainage for the embankment.

The bitumen content typically ranges between 6 to 6.5 %. Excess bitumen is usually added to the mix to allow for absorption. Modified paving machines are used to place, level and partly compact the core and adjacent gravel material. The mix is compacted with vibratory rollers at temperatures of around 160 °C to 180 °C. Final air voids for the mix after compaction are typically less than 3 %. This gives an almost impervious core.

In 2002 ATC design personnel visited Norway for discussions with Norwegian designers and contractors to investigate whether an asphaltic concrete core dam could be successfully constructed in Iran. It was concluded that an asphaltic concrete core dam could be constructed in Iran using hand placement techniques.

ATC became aware of an ACC dam being constructed in Iran and arranged to visit the construction site. The site is located on the southern coast of the Caspian Sea. The construction of this particular embankment was using hand placement techniques. The plinth can be seen in **Figure 2**.

Construction of an asphaltic concrete core dam at Miduk would most likely be undertaken by a contractor with no prior construction experience with ACC dams. To ensure the correct construction of such an embankment, strict testing codes have been recommended as part of the design. During the design process it was assumed that hand placement techniques would be used, however, a Norwegian group have since been contracted to oversee the construction and provide a core placement machine.

Figure 2 – View of Meijoran Dam, Ramsar

8.2 Material Selection

Critical to the design of an ACC is the asphalt mix and compaction. The asphalt mix must match Fuller's curve: this is the grading which provides the densest mix. Material is also tested for suitability based on inter-particle friction, durability, wettability, plasticity index, size and gradation **[Ref 7]**.

Local rock was tested as ACC aggregate and was found to be suitable.

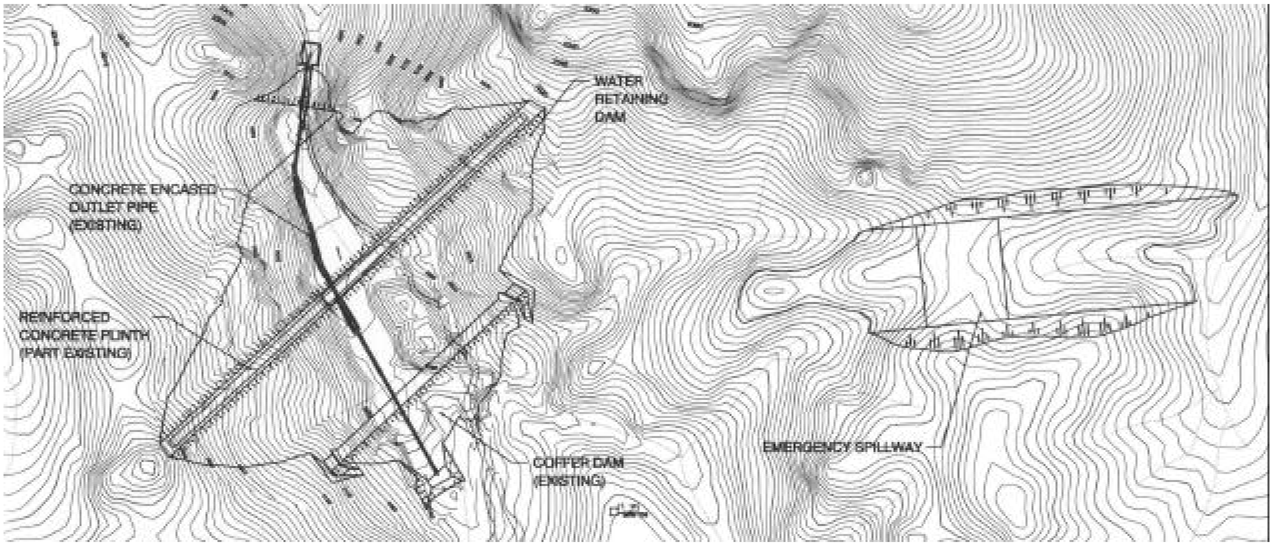

Figure 3 – General Layout

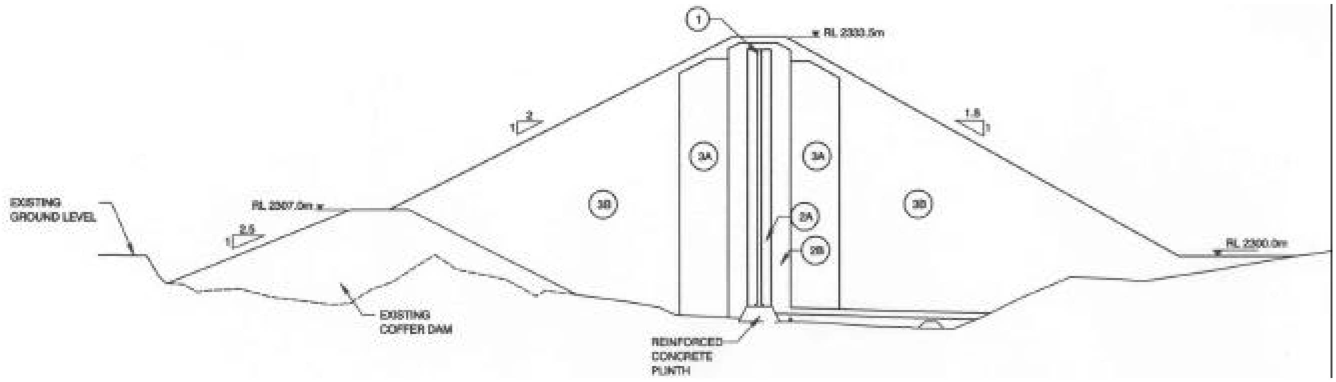

Figure 4 – Typical Section

9 DESIGN

The water retaining embankment was designed to store a 1 in 100, 72 hour storm rainfall event for the entire catchment (6.5 km^2). The rainfall intensity was estimated to be 4.8 mm/hr, and with initial and continuing losses, gave a required storage of 1.92×10^6 m^3. The spillway design level was set at RL 2329.5 m. The spillway was designed to the east of the storage, and with a 50 m base width, and 1:1 batters.

Freeboard on the embankment accounts for wave run-up, and reservoir setup due to wind, the design flood depth in the spillway, and an additional 0.5 m safety margin. A 4 m freeboard was adopted. The design embankment crest became RL 2333.5 m. The final embankment height will be 43 m.

9.1 Outlet Pipe and Plinth

A 500mm diameter outlet pipe was designed to pass through the embankment. The size of the pipe was determined by the client on the basis of make-up water requirements. The outlet pipe is designed to recycle water from the water retaining embankment to the plant via a return water pipeline. The outlet pipe is encased in reinforced concrete and is founded on rock.

The plinth forms the foundation for the ACC. It is constructed of reinforced concrete and is also founded on rock. The layout of the outlet pipe and plinth is presented in **Figures 3**.

9.2 Coffer Dam

The coffer dam is designed to RL 2307 m, and two spillways are constructed either side to RL 2305 m. Very little water is stored behind the coffer dam, it is constructed to provide protection to the works and to attenuate the flow of water during a storm rainfall event. The coffer dam spillways are designed for a 1 in 10 year rainfall event. The coffer dam is pervious, and is designed with a fine grained Zone 1 material and filter zones, with a rockfill outer shell. This shell would provide protection to the coffer dam should the embankment be overtopped during a storm event.

9.3 Foundation Grouting

Foundation grouting was designed to consist of contact, blanket and curtain grouting. Contact grouting is undertaken to ensure a good seal between the plinth and the foundations. Blanket grouting is to extend to 10 m below the plinth, and curtain grouting to 30 m.

9.4 Water Retaining Embankment

The water retaining embankment was designed with a 0.6 m wide ACC. The core is widened at the base and at the abutments to ensure good contact with the concrete plinth. The embankment was designed with a crest width of 8 m, and an upstream batter of 2:1 (Horizontal: Vertical), and a downstream batter of 1.8:1. A berm was designed on the downstream batter at RL 2300 m to provide protection to the outlet pipe from rock fall where is passes through a steep gorge, as well as provide additional stability to the embankment.

9.5 Material Zones

All materials used in the construction of the water retaining embankment are to be sourced from hard, durable slightly weathered to fresh rock. A typical cross section is presented in **Figure 4**. The material zones are presented in **Table 1**.

10 STABILITY ANALYSES

Critical sections were selected for the water retaining embankment. Stability was assessed for various water levels. The stability of the structure was analysed using GEOSLOPE software.

Table 1 – Embankment Material Zones

ZONE	TYPE	MATERIAL	PLACEMENT
1	Asphalt Concrete Core	Max. particle size 19 mm aggregate, matching fullers curve, and bitumen.	200 mm compacted thickness.
2A	Core Support	Max. particle size 75 mm, aggregate matching fullers curve, non plastic material.	200 mm compacted thickness, 6 passes of a 1.5 tonne (static) smooth drum roller.
2B	Transition	Max. particle size 37.5 – 150 mm.	400 mm compacted thickness, 6 passes of a 10 tonne (static) smooth drum roller.
3A	Rockfill	Blasted rock, maximum particle size 300 mm.	400 mm compacted thickness, 6 passes of a 15 tonne (static) smooth drum roller.
3B	Rockfill	Blasted rock, maximum particle size 600 mm.	800 mm compacted thickness, 6 passes of a 15 tonne (static) smooth drum roller.
4	Drainage Gravel	Nominal 6mm aggregate.	400 mm loose thickness, 10 tonne (static), compact with vibration off.

Material strength parameters used in the stability analyses were as follows:

Table 2: Material Strength Parameters

Zone	Description	Depth (m)	Cohesion (kPa)	Effective Angle of Friction
1	Asphaltic Concrete		2000	-
2A	Granular Fill		-	38°
2B, 3A, 3B	Rockfill	0 – 8.5 8.5 – 22 22+	- - -	53° 49.5° 47.3°
4	Granular Fill (Drainage zone)		-	36°

The strength of rockfill is dependant on effective confining pressure, particle size distribution, strength of particles and degree of compaction **[Ref. 5]**.

10.1 Factor of Safety against Sliding

Factor of Safety (FOS) against sliding was assessed and found to be approximately 2.

10.2 Static Stability

The static stability for circular failure was 2.6 for the critical upstream case, and 2.4 for the critical downstream case.

10.3 Seismic stability

10.3.1 Operating Base Earthquake (OBE)

The FOS was assessed for the OBE using the Corps of Engineers pseudo-static method **[Ref. 8, Ref. 9]**. This procedure sets the earthquake coefficient to 50% of the design acceleration, and drained strengths used are reduced to 80% of their original value. The method states, that for a FOS of one or more, the maximum allowable deformation is one metre. The results of the analyses are presented in the table below:

Table 3: OBE Stability Results

Case	Water Level	FOS
Downstream	Maximum+	1.46
Upstream	Maximum+	1.27
	Half Height	1.29
	Operating Height*	1.34

*Crest Level of Coffer Dam

+Full supply Level plus one metre

10.3.2 Maximum Design Earthquake (MDE)

The amount of deformation that occurs during the MDE was calculated using the technique presented by Makdisi and Seed **[Ref. 9]**. This technique requires the embankment response to the MDE, in particular the crest acceleration and natural period. These numbers were determined by Dr. E. Kavazanjian Jr. **[Ref. 4]**.

The maximum crest acceleration, Ümax is 1.63g and the period, T_0 is 0.83 seconds. The procedure generally is as follows: a failure surface is selected, the yield acceleration for a FOS of one calculated, and the movement calculated based on the relationships presented in Makdisi and Seed **[Ref. 10]**.

Trial surfaces analysed passed through the core and crest. A range of surfaces were analysed at various depths and water levels. The resulting displacements ranged from 0 m to 0.75 m. The maximum displacement was for a trial failure surface on the upstream face passing through the coffer dam, with a maximum water level.

This displacement is expected to occur as a zone of deformation. The major advantage of an ACC embankment is that the core is self healing. Because of this, ACC embankments are well suited to seismically active areas.

11 CONSTRUCTION

Up to April 2004, all of the foundation stripping and part of the foundation preparatory works have been completed. Foundation stripping involved the removal of all the alluvial and colluvial materials, and completely weathered rock. The exposed foundation materials consisted of moderately weathered to faintly weathered and fresh andesite. Stripping was undertaken during the design process.

The outlet pipe is complete, more than half of the plinth has been constructed, and grouting is underway. The coffer dam has been constructed to protect the works during the wet season including emergency spillways. The plinth and some of the foundation stripping is can be seen in **Figure 5**.

12 CONCLUSIONS

Asphaltic concrete embankments have advantages in countries where water and clay is scarce and bitumen is readily available. The costs of constructing an ACC embankment are comparable to a concrete deck and clay core embankment. For seismically active areas such as Iran, an ACC embankment is ideally suited as it is self healing.

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Figure 5 – Water retaining embankment (under construction) March 2004.