The Staged Construction of Imang Dam

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SYNOPSIS. Imang Dam and reservoir was constructed between 1995 and 1997 for irrigation purposes and is located on the Sg[1] Imang in the Brunei-Muara District of Brunei Darussalam. The dam has a maximum height of about 10m, length of 420m and contains a reservoir of $10Mm^3$, which is supported by a $14km^2$ secondary rainforest catchment. The scheme was originally designed to serve local irrigation requirements of about 700 ha of double cropped rice.

A major feature of the design of the embankment dam was the soft clay foundation and wide stability berms. The dam was originally designed to be constructed in one stage following treatment of the foundation soils to accelerate consolidation settlement and increase shear strength. However, the foundation materials did not gain sufficient strength during construction to allow the embankment to be constructed to its full height and interim measures were implemented to allow first filling and operation of the reservoir. Following a period of foundation consolidation settlement and strength gain, the embankment crest was raised to its full height in 2006. This paper describes the design and staged construction of Imang Dam.

BACKGROUND

Imang Dam was first proposed in 1975 to support irrigation of the Mulaut area for mechanised cultivation of rice padi. The first series of field investigation and feasibility studies were carried out by MWH by 1982 and these early studies highlighted the difficulties in constructing a dam embankment of more than about 6m in height due to the deep estuarine and swamp deposits covering the dam site. Further investigation and studies were concluded in 1990 and the construction of the dam and reservoir works commenced in 1995.

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[1] Sungai (Sg) – river or stream

Ensuring reservoir safety into the future. Thomas Telford, London, 2008

MAIN FEATURES OF THE SCHEME

Imang Dam is formed by a 10m high earthfill embankment with wide stability berms. The Full Supply Level (FSL) is 11.5m BSD (Brunei Standard Datum) and the nominal dam crest level is 13.9m BSD. The foundation comprised soft to very soft alluvium, swamp deposits and soft estuarine clays to depths of almost 28m and it was neither technically nor economically feasible to remove and replace these soft materials. A review of embankment dams on soft ground was carried out and a wide bermed embankment design was developed that incorporated much of the experience gained by MWH in Brunei during the 1980s at Benutan Dam.

The dam is approximately 420m long and incorporates a 35m wide ogee shaped overflow / spillway and outlet structure at the right abutment ridge. A further auxiliary spillway with a 55m wide overflow weir is located further along the right abutment ridge to cater for flood events in excess of 200-year return period. The combined overflow system is designed to pass the PMF discharge of 383 cumecs and the arrangement was tested by a physical hydraulic model.

The outlet structure was used for diversion of the Sg Imang during construction of the embankment and later fitted with a series of four penstock gates to control irrigation water into a canal distribution system, which was to convey water to the irrigation areas. However, the scope of the canal system was curtailed and a recent scheme has installed a dedicated pipeline feeding the nearby plantations that have been developed. The general arrangement of the dam and ancillary works is shown on Figure 1.

DAM SITE GEOMORPHOLOGY AND GEOLOGY

The dam site is situated in the Bukit Siudam range of hills and lies on the Miri Formation rocks comprising laminated siltstones, silty mudstone and fine-grained sandstones of marine origin. The beds lie on the western flank of the Jerudong anticline and dip at up to 40° adjacent to the Mulaut plain. The topography is controlled by the structure of the underlying Miri Formation rocks. The beds strike uniformly at N18° E forming a series of parallel strike ridges and cuestas (dip slopes). Faulting is shown on the published geological map of the area but there was no evidence for any major fault in the immediate vicinity of the dam site or reservoir basin.

The valley floors consist of alluvium occupying deeply incised stream channels that were eroded during the Pleistocene glacial maximum and subsequently filled with sediment as the sea level rose to the post-glacial maximum of about 1.8m BSD. Estuarine alluvium was deposited at the peak of the marine incursions over extensive areas of the Brunei valley including the Mulaut plain.

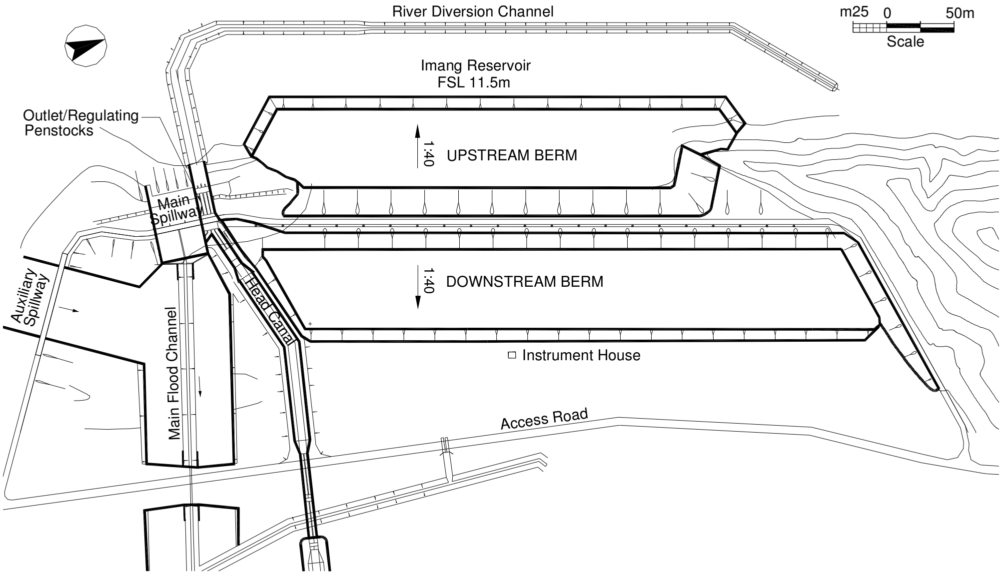


Figure 1 General Arrangement of the Dam and Ancillary Works

The abutments to the dam are provided by narrow ridges of sedimentary rock, which follow the strike of the strata, SSW to NNE. The valley floor between the ends of these ridges is formed by a broad flat alluvial floodplain 290m wide interrupted by a 30m wide outcrop of fine grained sandstone. Two river channels pass one to either side of this outcrop and meet about 80m downstream of the dam.

The maximum depth of alluvium encountered in the boreholes beneath the left and right hand channels was about 21m and 28m respectively. The superficial soils infilling the valley floor were classified into three distinct strata in descending order of depth were:

Riverine Alluvium: Generally loose, clayey sandy silt and very soft, silty sandy clay, of low to intermediate plasticity. Average thickness of 3.4m (2.4m to 4.3m thickness).

Swamp Deposits: Generally very silty, soft to firm, brown grey to dark grey, clay of high plasticity with abundant fragments of decaying wood and lenses of peat. Average thickness of 5.5m (2.4m to 8.8m thickness).

Estuarine Sediments: Generally very silty, sometimes very sandy, soft to firm, brown grey to dark grey, clay of variable plasticity (high at the top of the layer and reducing to low at depths of 16m and more below ground). Average thickness 8.5m (up to 14.9m thick).

The valley sides are generally covered by a mantle of colluvium consisting of sandy clay and silty clays to depths of about 1m. Bedrock consists of Miri Formation of laminated brownish grey siltstones, grey silty mudstones and fine-grained sandstones. The beds dip westward (upstream) at angles between 30° and 40°. The beds strike uniformly at N 18° E. The simplified geological section through the dam is shown on Figure 2.

Site investigations were carried out during the feasibility stage in 1980 and at the design stage in 1989/90. Both investigations included boreholes, trial pits and laboratory testing. The design stage investigation concentrated on establishing reliable in situ shear strength data for foundation shear strength estimation and included in situ vane and self-boring pressuremeter tests.

EMBANKMENT DAM DESIGN

The design concept for the dam was dictated by the strength and thickness of the alluvial soils, which occupied the valley floor. The alluvium was soft, weak and highly compressible and the preliminary design considered two options for construction of the dam embankment. These were:

Option 1: excavation of the alluvium down to bedrock and replacement and compaction of suitable materials.

This option would have entailed excavation of the alluvium to bedrock and replacement and compaction with suitable materials below the groundwater table. The depth of the weak alluvial soils was prohibitively deep (up to 28m) and the volume of foundation material would have been greater than the embankment fill volume by ten-fold. Also, the success of a dewatering scheme required to facilitate such a design was also questionable. Therefore, this option was ruled out on cost and technical grounds.

Option 2: Construction of an embankment on the alluvial foundation using wide stability berms and foundation treatment to accelerate consolidation of the weak soils.

This option entailed stripping the upper surface of the alluvium and construction of a wide bermed embankment directly on the existing foundation materials. This design concept had been successfully used at Benutan Dam and incorporated 60m wide upstream and downstream stability berms. The alluvial soils would be improved by installing pre-formed ‘wick’ drains under the central section of the dam to improve drainage and accelerate consolidation settlement, thereby improving the strength of the weak foundation. The preliminary design developed these concepts and was later validated using finite element techniques.

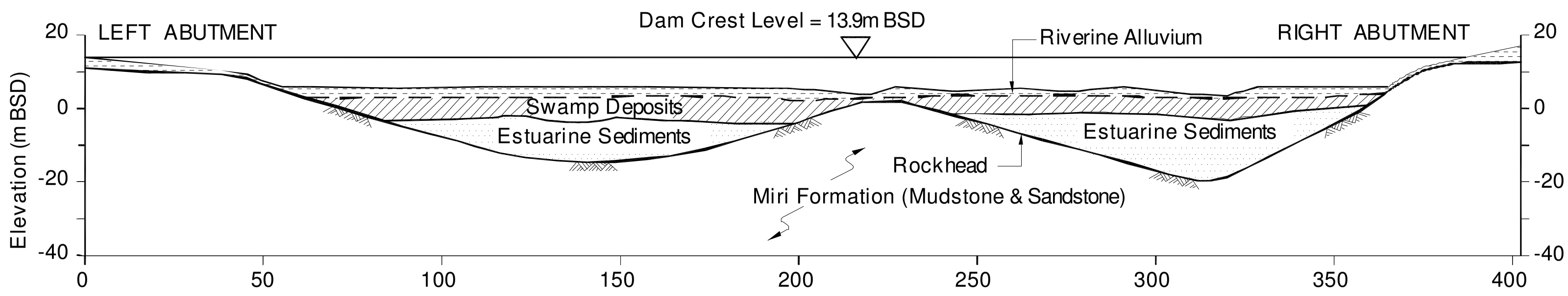


Figure 2 Simplified Geological Section along Dam

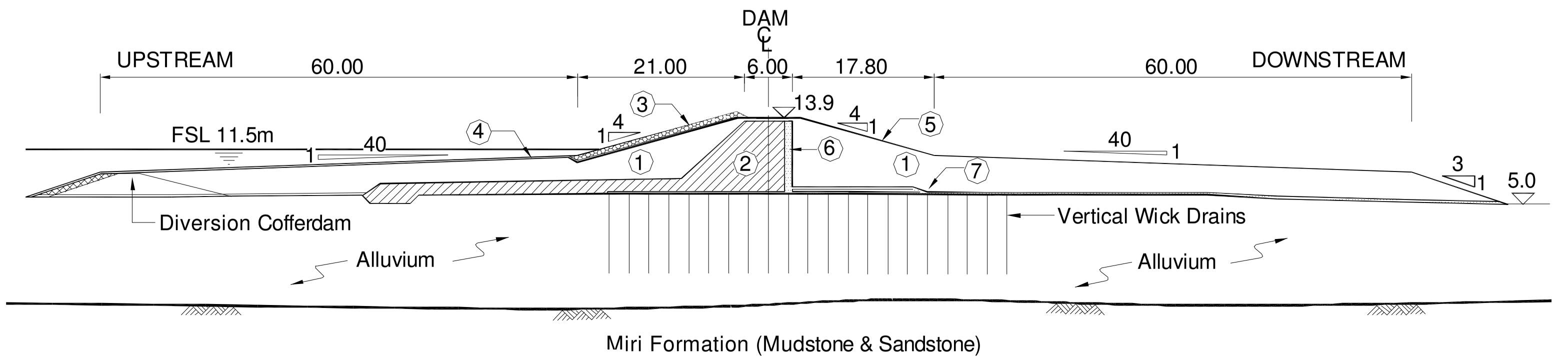


Figure 3 Cross section of Dam

Legend: (1) Shoulder Fill (2) Clay Core / Clay Blanket (3) Rip Rap Slope Protection (4) Beaching Slope Protection (5) Topsoil and Turf (6) Wall Drain (Fine Filter) (7) Drainage Blanket & Finger Drains

Preliminary Design
The dam section was initially designed by limit equilibrium stability analysis using undrained parameters for the foundation soils. Measurement of undrained shear strength of soft and very soft clays is difficult in practice and a number of in situ and laboratory tests were conducted as part of the ground investigations to estimate the relationship between undrained shear strength and depth through the alluvium (i.e. in situ vane shear, self boring pressuremeter, undrained triaxial tests). These were also compared with empirical relationships such as that proposed by Mesri (1975) for normally consolidated clays, $C_u = 0.22\sigma_{p'}$, where $\sigma_{p'}$ is the preconsolidation pressure.

The undrained shear strength profiles derived are shown on Figure 4. The design profile (1) was adopted for initial design purposes and was reviewed and further modified (2) as part of the Finite Element analysis (described below) to incorporate additional ground investigation data and a critical review of the test procedures and results. The shear strength of the alluvium was further estimated during construction (3) using the Mesri relationship and observations of pore water pressure from the foundation piezometers.

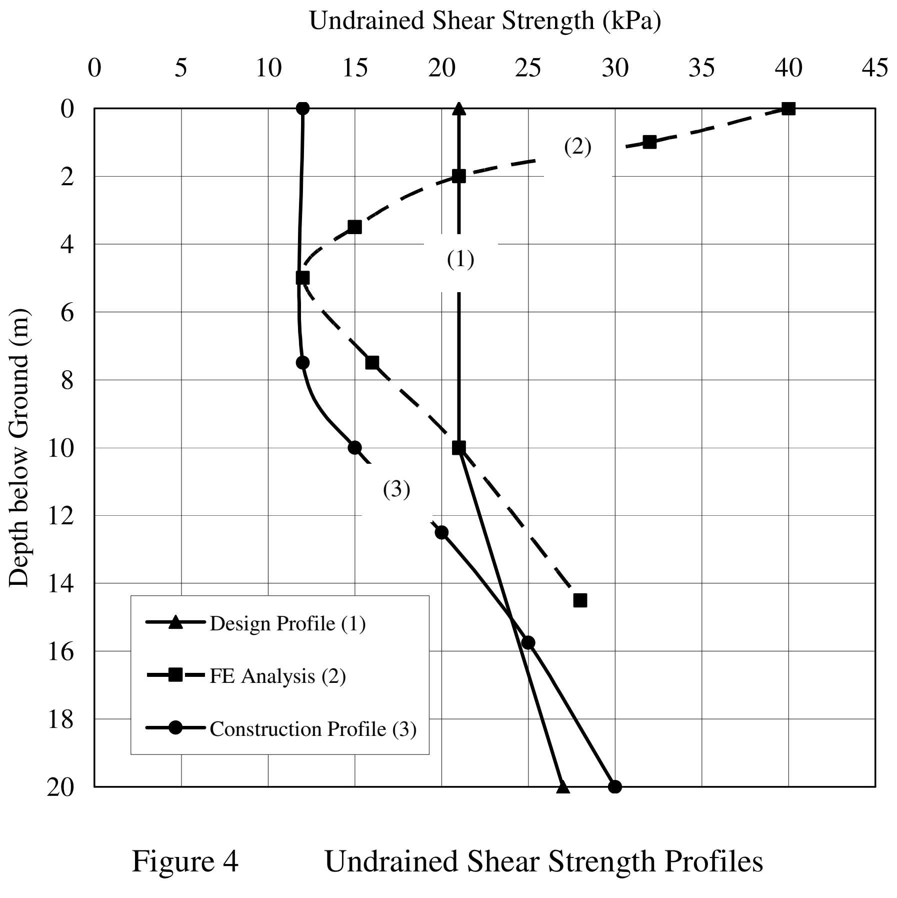


Figure 4 Undrained Shear Strength Profiles

The embankment fill material effective strength parameters of c'=0, ϕ'=30° and bulk density of 20 kN/m^3 were adopted in the initial design and a conservative pore pressure ratio (pore pressure/weight of fill above), r_u of 0.4 was assumed. The results of the stability analysis indicated that stability during construction would be the critical condition with the lowest factor of safety calculated for the upstream shoulder.

Factors of safety would improve following reservoir impounding in the long terms as the foundation gained strength through dissipation of porewater pressures and consolidation settlement. A range of dam crest heights, berm widths and berm height configurations were investigated, with the final arrangement shown on Figure 3. Both upstream and downstream berms were 60m wide with the diversion cofferdam formed in the upstream berm.

Finite Element Analysis

The limit equilibrium stability analysis indicated that the dam section would be satisfactory at the end of construction, but did not model the effect of any dissipation in excess pore pressures in the foundation soils or any strength gain during construction.

Finite Element (FE) methods were therefore used during the detailed design stage to review the soil properties, geometry and predicted performance during construction. Geotechnical Consulting Group (GCG) carried out this work using the Imperial College Finite Element Program (ICFEP).

The shear strength of the alluvium had the overriding effect on stability of the embankment and the shear strength profile adopted in the FE analysis was modified following further review of the original test data and additional in situ measurements carried out during the detailed design stage. The revised profile is shown on Figure 4 and has a minimum shear strength of 12 kPa at about 5m depth. This profile was considerably lower than that assumed in the preliminary design (minimum: 21 kPa) but, given the difficulties in obtaining reliable measurement of shear strength in very soft clays and the implications for stability, is was considered to be a safe and reliable assumption.

The analysis showed that vertical displacement under the central section of the dam would translate to horizontal displacement under the upstream and downstream berms with lateral spreading of the foundation / embankment. The central 'pimple' of the embankment was expected to cause local stress concentrations in the foundation below the central and upstream section. Foundation drainage was therefore provided in this location in the form of vertical 'wick' drains in order to improve drainage, accelerate foundation settlement and increase foundation shear strength.

The finite element studies concluded that the embankment could be constructed in a single lift, although there would be little margin against failure at the end of construction. The computed factor of safety at end of construction would be about 1.1 and following impounding of the reservoir would increase to about 1.6 in the long term. Stability during construction would therefore be marginal and an observational approach was adopted based on an extensive instrumentation scheme. The final geometry of the dam embankment and foundation treatment is shown on Figure 3.

EMBANKMENT MATERIALS

The embankment incorporates a central rolled clay core and upstream blanket as the waterproof element with earthfill shoulders and wide berms. The colluvium and highly weathered mudstone rock members were used to form the clay core and blanket, following some upward adjustment in moisture content so as to produce a dense fill with 98% relative compaction and a shear strength of between 70 and 100 kN/m^2. This required placement moisture content in the range 21-25%, which was generally a few percent above the natural moisture content in the borrow areas.

The core material was watered, leveled in 250mm thick layers and compacted using pad foot compactors and produced a satisfactory dense fill with less than 4% air voids.

The shoulders were formed from weak rock members of the Miri Formation arising from the required excavation for the diversion / spillway structures and also from a borrow area located within the reservoir basin. The mudstone broke down readily and was compacted using pad foot compactors, while the sandstone members were compacted using smooth vibratory drum rollers.

There were no suitable local sources of graded granular materials or rockfill materials for the internal filters or rip rap slope protection and these were all imported to the site. The ‘critical filter’ wall drain located immediately downstream of the core was connected to a base drainage blanket, which extended to the toe of the central downstream slope. This was a ‘sandwich’ construction of coarse drainage material within fine filter that drained under the downstream berm via a series of ‘finger’ drains of similar construction.

The foundation under the central and upstream section of the dam was treated by installation of a grid of vertical ‘wick’ drains taken to 10m depth on a 2.5m grid spacing. These consisted of a proprietary corrugated plastic internal member wrapped with geotextile filter fabric. The heads of the wick drains were connected to a sand drainage blanket at ground level to provide a drainage outlet and reduce seepage path lengths.

EMBANKMENT PERFORMANCE DURING CONSTRUCTION

The dam was instrumented along two principal sections with a total of forty-four vibrating wire piezometers in the foundation and twenty-two in the embankment. Settlement and displacement was also monitored by a series of nine combined inclinometer/extensometer tubes set into bedrock.

A trial embankment was constructed as part of the upstream diversion cofferdam and was later incorporated into the upstream berm. This enabled the performance of the 'wick' drains to be observed. These trials showed that areas adjacent to the 'drained' areas did not drain or consolidate and cracks formed at the toe of the embankment. The final wick drain spacing was adjusted with the deepest closer spaced drains under the highest section of the dam.

Porewater pressures were monitored as the fill levels increased and piezometric pressures responded closely to fill height, as expected. Excess pore pressures within the foundation increased with depth with the 'wick' drains being particularly effective at shallow depth (less than about 5m depth). Calculated pore pressure coefficients, r_u, at depths greater than about 5m depth were variable and ranged between r_u = 0.55 and r_u=0.85. Typical plots from the piezometer readings are shown on Figure 5.

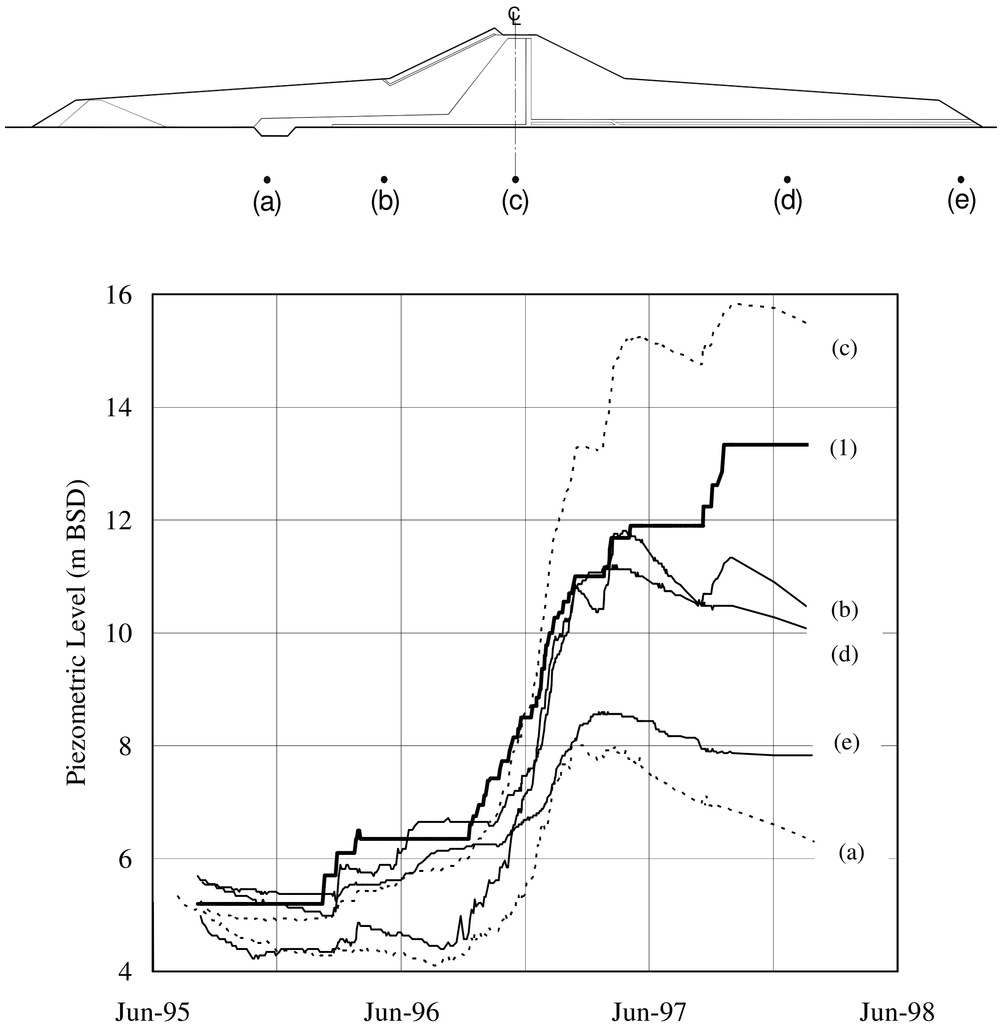
Total settlement during embankment filling was not excessive and ranged from about 300mm under the downstream berm to 700mm under the dam crest (an indication of limited pore pressure dissipation). Horizontal deformations were up to 170mm in the downstream direction and were greater than predicted by the finite element analysis.

The Contract included provision for a shutdown period to allow the instrumentation to be monitored when no additional fill material was placed to assess the performance of the dam and its foundation. The fill level reached about 12m BSD, 7m above foundation level, when the shutdown period was invoked. The piezometers located within the drained section showed quicker dissipation of porewater pressure (approximately 7mm to 13mm per day), than those located in the undrained sections (less than 5mm per day).

The conclusion drawn from the instrumentation data was that the loads imposed on the foundation clays by the dam was mobilizing a greater proportion of the available strength than had been expected. This warranted a cautious approach to completion of the embankment.

Following discussions with the Client, it was agreed that the embankment should be filled to 13.25m level (approximately 2m short of the design height at the end of construction) with the remainder of the dam crest completed at a later stage, when the foundation pore pressures had dissipated sufficiently to increase strength of the foundation.

The remaining fill was placed in a strictly controlled manner with limitations placed on the contractor to place fill to a specific zoning sequence. Monitoring was continued after placement of fill in each zone and results provided to the designer before proceeding with subsequent layers.



Legend:-

(1) - Fill Level at Dam Axis

(a) to (e) – Foundation Piezometer Locations

Figure 5 Foundation Responses to Embankment Loading

INTERIM DAM CREST ARRANGEMENT

A review of the flood hydrology was carried out with the Full Supply Level reduced by 220mm to 11.28m BSD by omission of the pre-cast overflow crest blocks (giving a gross freeboard of 1.97m). The flood capacity for the dam with the dam crest at 13.25m BSD was estimated to be about 0.5 PMF, which was deemed to be acceptable as a short term measure.

In order to limit the load applied to the foundation a revised crest detail was developed, as shown on Figure 6, which provided for raising the crest to its design level in the future. Core material was stockpiled for later use and the works were finally completed and reservoir impounded in 1998.

FINAL DAM CREST ARRANGEMENT

Monitoring of the instrumentation was not continued by the owner after commissioning of the reservoir to its interim level in 1998, although data from the piezometers and settlement instruments was obtained in 2004 prior to procurement of the works to complete the dam crest.

This information indicated that settlement had been substantial (in the order of 1.0m) and foundation pore pressures had reduced by between 4m and 7m. Therefore, it was clear that pore pressures in the foundation had dissipated and significant consolidation settlement had occurred and was now safe to raise the embankment to its original design height of 13.9m (min). A settlement allowance of up to 500mm was provided where the foundation soils were thickest to allow for future settlement and the embankment was constructed to an inverted ‘w’ profile to match the foundation thickness with a maximum construction level of 14.4m BSD. The details of the crest raising are shown on Figure 7.

The existing dam crest level was stripped and leveled and the core zone was exhumed to assess its condition. A key trench was excavated and the core material was found to be in good condition. No sign of desiccation of the existing core material was evident. The core and wall drain were raised to the same specification as the original works, with the upstream slope protection re-graded and extended to the new dam crest level. The reservoir level was drawn down to approximately 7.5m BSD during the dam crest raising works. The completed embankment is shown on Figure 8.

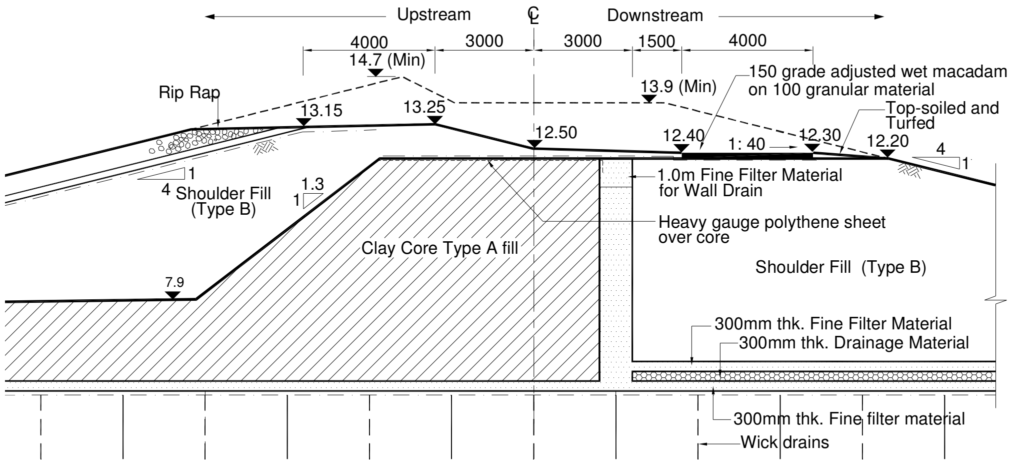


Figure 6 Interim Dam Crest Detail

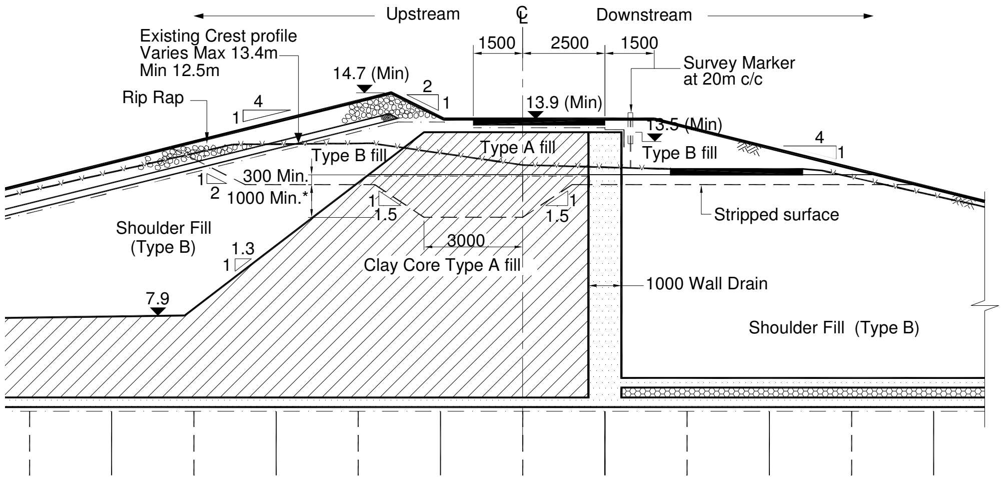


Figure 7 Final Dam Crest Detail

Figure 8 View of Completed Embankment in 2006.

CONCLUSIONS

Imang Dam is formed on a weak and highly compressible clay foundation and special design features and close construction monitoring were employed to ensure that the embankment did not fail during construction. In particular, the following points are noteworthy:

i) The embankment incorporated wide berms to counteract rotational failure through the soft alluvial foundation. Drainage conditions within the central part of the upper alluvium were improved by installing a series of vertical 'wick' drains. These were useful in dissipation of excess pore pressures beneath the central part of the dam.

ii) Measurement of shear strength of weak clays for use in stability analysis is difficult in practice - especially in hot tropical jungle environments. A number of in situ tests, laboratory tests and empirical estimates were carried out at Imang Dam to derive a shear strength profile for the foundation materials. The final profile adopted for design was biased towards the in situ vane test results and the Mesri formula.

iii) The instrumentation data obtained during construction indicated that the foundation pore pressures were high and showed little or no dissipation, other than in the drained sections. It was concluded that the load imposed by the embankment on the clay foundation was mobilizing a greater proportion of the available strength than had been expected and completion of the embankment in a single stage would not be advisable. Following a shutdown period to review the performance of the foundation with no additional loading, the embankment crest was finished some 2m short of its design height and the spillway crest blocks omitted to maximize flood freeboard.

iv) Although the embankment was not completed in a single stage as originally planned, the interim crest details and omission of the spillway crest blocks provided acceptable flood protection in the short term (approximately 0.5 PMF) and enabled the reservoir to be safely impounded to near its design level.

v) Following a period of operation of the reservoir, the foundation pore pressures dissipated considerably resulting in large settlements of up to 1.0m, where the foundation thickness was greatest. The dam crest was raised in 2006 by 2 - 2.5m to its design level and the spillway crest blocks installed bringing the reservoir flood protection to the PMF level.

ACKNOWLEDGEMENTS

The authors wish to extend there thanks to the Department of Water Services, Public Works Department, Brunei Darussalam for permission to publish this paper. Thanks are also due to Professors Vaughan and Hight of Imperial College, London and to their colleagues at GCG.

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