DESIGN AND CONSTRUCTION OF A PORT IN SOUTHEAST MADAGASCAR

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A new deepwater port, Port d'Ehoala, is being constructed at the southeastern tip of Madagascar in order to facilitate the export of ilmenite, a heavy mineral, via 60,000 DWT ships, as well as to accommodate the import/export of other commodities to/from this remote region. Baird was responsible for the planning, design, and construction of the port, which incorporates a 600 m long breakwater, a multi-use quay, dredging, land reclamation, and a groyne. The project site is challenging in terms of the environmental conditions, as well as the issues inherent in working in a remote region. The wave climate is severe and bi-modal in nature, with intermittent easterly seas superimposed over persistent southerly swells (Hs = 2 to 3 m is typical). In addition, storm waves in excess of Hs = 8 m are possible during a tropical cyclone. The severe wave climate dictated the requirement for a substantial breakwater to provide protection to the port. In addition, rapid advancement of breakwater construction was critical to the overall project schedule, as protection was required to facilitate dredging, land reclamation and quay construction. This paper will provide an overview of the development of Port d'Ehoala, with emphasis on the design and construction of the breakwater.

INTRODUCTION/BACKGROUND

QIT Madagascar Minerals' (QMM) mineral sands mining project, in the Fort Dauphin region of southeastern Madagascar, is the largest investment in the island's history (US$660 million). Rio Tinto owns 80% of QMM through its subsidiary QIT; the Madagascar Government owns the other 20%. Production of ilmenite (titanium dioxide) from the mine is expected in late 2008. This project will be the catalyst for broader economic development of the country, while also providing environmental conservation opportunities. Additional information on the project may be found at http://www.riotintomadagascar.com.

A key component of the project is a port to facilitate loading of ilmenite sand onto bulk carriers for export. Initial studies, dating back to the late 1980s, considered alternative sites for the port, as well as various port concepts.

QMM, in cooperation with the Madagascar Government and the World Bank, developed a legal and fiscal framework for the project that includes a sheltered, multi-user, deep-water port facility. The US$145 million port will meet QMM's requirements (initial ilmenite production of 750,000 tonnes/year, with potential expansion to two million tonnes/year), but will also provide facilities for the import/export of other commodities by third party users. Over time, it is expected that the port will make an important contribution to the economic development of the entire region.

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Consideration of numerous factors led to the selection of Pointe Ehoala, at the south end of Fausse Baie des Galions, as the site of the new port. The site location is presented in Figure 1.

Figure 1. Site Location – Fausse Baie des Galions

Baird & Associates (Baird) was retained by QMM to undertake the planning and design of the port, and to oversee its construction. The port layout was developed based on a thorough assessment of functional requirements, as well as detailed investigations and analyses of the site and environmental conditions. A schematic overview of the port layout is presented in Figure 2.

Figure 2. Schematic Overview - Port d'Ehoala

The key features of the project include the following:

- 600 m long breakwater extending into 15 m water depth;
- Multiuse quay, including 275 m long primary berth for bulk carriers up to 60,000 DWT and 150 m long secondary berth for third party users;
- 783,000 m^3 of dredging for harbour basin and entrance channel;
- 34.5 hectares of land reclamation;
- Groyne to limit sedimentation in the dredged area;
- Landside port facilities (cargo handling and storage) and infrastructure.

SITE CONDITIONS

As shown in Figure 1, the project site is located at the south end of Fausse Baie des Galions, a large bay anchored by Cape Ranavalona to the southwest and Cape Antsirabé to the northeast (where the town of Fort Dauphin is located). The shoreline within the bay consists of wide, sandy beaches backed by large sand dunes. The beaches become relatively narrow as one approaches the project site, with scattered rock outcrops along the shoreline at the site. The Ehoala headland (and adjacent shallow rock shelf and small islands) is a dominant feature at the project site, extending towards the NE off the Cape.

Extensive site investigations were undertaken in 2003-04, including topographic, beach and hydrographic surveys, and geophysical investigations. Unfortunately, it was not possible to collect geotechnical information (i.e. boreholes), as the mobilization cost to get a jackup barge to the site was in excess of US$1 million. The geophysical information was verified to the extent possible using diver inspections, underwater test pits and land-based cores. However, the subsurface conditions represented a key uncertainty during the design process, particularly for the quay structure and dredging.

Figure 3 illustrates the bathymetry within Fausse Baie des Galions. Key features of interest include the shallow shelf feature adjacent to the Ehoala headland, as well as a series of shoals (reef outcrops) scattered around the bay and beyond. Interpretation of the geophysical data indicated a complex, layered geology at the port site. The shallow shelf feature was determined to be a combination of aeolianite and beach rock overlying sand with cemented layers, while the adjacent area (to the north) was determined to be loose sediments.

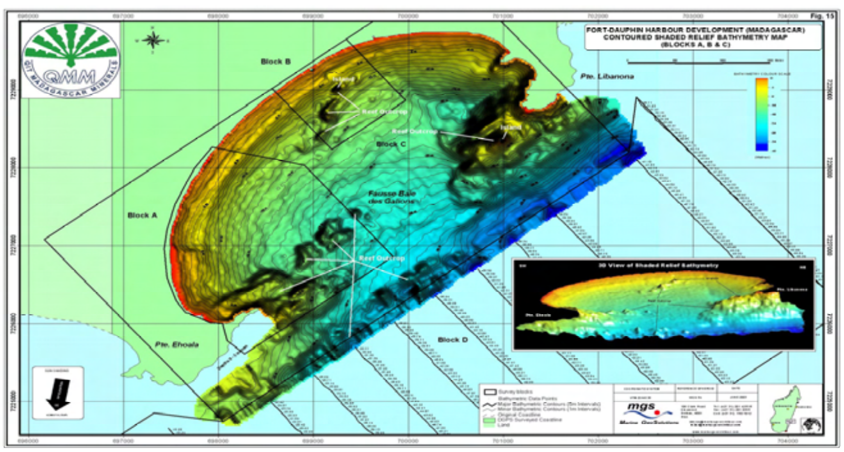

Figure 3. Bathymetry within Fausse Baie des Galions

PORT LAYOUT

The bathymetry and subsurface conditions were important considerations in the development of the port layout. In particular, the Ehoala headland and adjacent rock shelf and small islands provide a certain level of natural shelter to the south end of the bay from the persistent southerly swell waves. Given the shallow depths in this area, this was a natural starting point for the breakwater. In addition, the aeolianite/beach rock feature was considered to provide a good foundation for the quay structure, while the adjacent area of loose sediments would be easier to dredge. Further, a berth aligned along the aeolianite/beach rock feature would be parallel to the predominant wind direction, a benefit with respect to the departure of fully laden ships. These factors, among others, led to the development of the port layout shown earlier in Figure 2.

The uncertainty related to the subsurface conditions (i.e. due to the absence of geotechnical information) was a key uncertainty/risk for the design of the quay structure. This issue was addressed as follows:

- Quay to be bid as a design-build component;
- Assumed soil conditions presented in Bid Documents:
- Contractor to do geotechnical investigation as early stage of work;
- Contractor to develop final design for quay structure;
- Baird to review and approve final design;
- Negotiation to address cost implications of change in soil conditions.

This approach was deemed to be the best way to address the uncertainty/risk noted above, and also allowed individual Bidders to propose different design concepts that take advantage of their varied project experience and equipment.

In addition, there was some uncertainty regarding the sediment transport processes in the bay, due to the complex (bi-modal) wave climate and the lack of historical information on shoreline variability. As such, a groyne was incorporated in the project design (north of the dredged harbour basin), with the length of the groyne to be established by beach monitoring during construction.

DESIGN CONDITIONS

The wave climate at this site is characterized by persistent southerly swells (Hs = 2 to 3 m is typical), with intermittent easterly seas. In addition, the region is subject to the effects of tropical cyclones, with the potential for very severe wave conditions, though such events are rare (the last one being a "near miss" by Deborah in 1975). The tidal range is relatively small (less than 1 m).

Extensive wave climate investigations were undertaken to define the wave climate at the site. A long-term hindcast (1985-2004) was undertaken for the entire Indian Ocean in order to simulate the generation and propagation of swells (generated by storms in the 40-60°S latitudes) and local swells. The WaveWatchIII (WWIII) model was utilized for the hindcast. The over water wind field was defined using the NCEP/NCAR reanalysis winds, including spatial and seasonal adjustments based on comparison to QuikSCAT satellite scatterometer measurements. The wave climate was validated against 15 years of satellite altimeter measurements, and 12 months of buoy measurements at the site. Figure 4 presents sample output from the WWIII model.

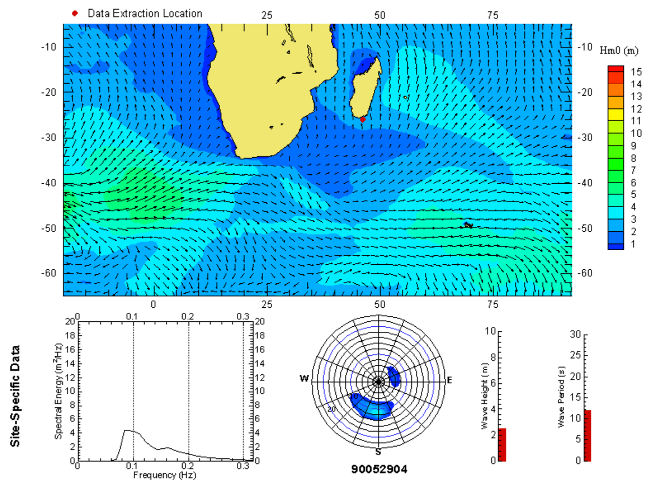

Figure 4 – Sample Output from WaveWatchIII Model

Nearshore wave transformations were simulated using the MIKE21 Nearshore Spectral Wave (NSW) model. The 20-year nearshore wave climate (in a depth of 21 m) is summarized below:

- Peak wave direction typically East through SSE;
- Mean Hs/Tp= 2 m/11 s;
- Max. Hs = 6.2 m (with Tp = 12 s);
- Max. Tp = 22 s (with Hs up to 3 m).

A higher resolution wave model was used to simulate waves and surges generated by tropical cyclones (TCs). Given the scarcity of data regarding TCs in this region (storm tracks were available back to 1945, but information on storm severity was only available for a few events), a Monte Carlo simulation was utilized, with a database of 1,000 "synthetic" TCs developed and simulated in the wave model. Analyses of these data indicated that the extreme wave conditions would be depth-limited at the project site. MIKE21 NSW simulations (and physical model tests undertaken later) indicated breaking wave heights of up to Hs = 8 m (Tp = 10 to 16 s) along the outer end of the proposed breakwater.

QUARRY INVESTIGATION

The identification of a suitable source of quarried stone (for coastal structures) and aggregates (for roadways and concrete) was a key task. Initial reconnaissance around the project area identified a number of potential quarry sites. Preliminary evaluations identified the "North-South-West hill complex", located approximately 10 km from the port site, as the most likely source for this project (refer to Figure 5).

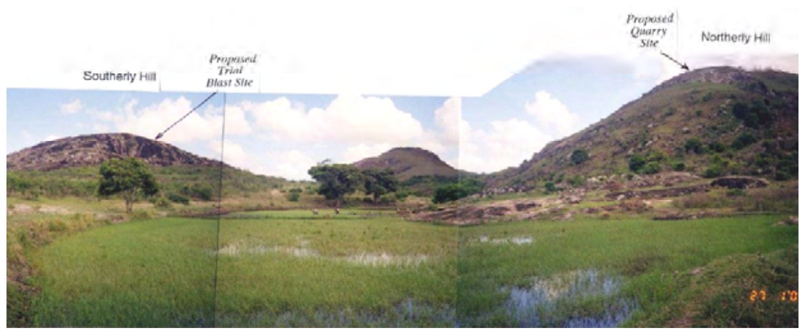

Figure 5 – Overview of Proposed Quarry Site

A preliminary test blast was undertaken in 2003, and raised concerns regarding the ability to produce large armour stone. As a result, initial design development for the breakwater assumed concrete armour units (see below).

More detailed quarry investigations were undertaken in 2004, including surface outcrop and scan-line mapping, coring, sampling and testing programs, and fragmentation and yield analyses. These investigations identified a wide range in stone materials in the N-S-W hill complex, with quality ranging from marginal to excellent. In addition, the fragmentation and yield analyses indicated that there was a sufficient quantity of stone available to meet the needs of this project (in fact there was excess capacity). In addition, it was concluded that the upper limit on armour stone size would be limited by production methods and equipment rather than by inherent limitations in the in situ rock. Based on this information, detailed design development for the breakwater focused on a berm design, assuming a maximum stone size of 20 tonnes (see below).

EVALUATION OF ALTERNATIVE BREAKWATER DESIGN CONCEPTS

As noted above, initial design development for the breakwater assumed the use of concrete armour units due to uncertainty related to the availability of large armour stone. Single layer units, such as the Accropode™ and Core-Loc™, were identified as the most "efficient" approach to resist the design wave conditions. However, the requirement to place the units in persistent swells, as well as the large size of the units, raised two significant concerns:

1. Ability to achieve required placement in swells (filter and armour);
2. Risk of breakage during placement (and in service).

The first issue was addressed in a set of two-dimensional model tests designed specifically to assess the impact of placement in waves on the stability of the armour layer. These tests, undertaken by HR Wallingford, included tests with both Accropodes™ and Core-Locs™ (12 and 16 m^3 units), as well as tests with various placement scenarios, including:

- Individual placement "in the dry" (typical in models, but not realistic);
- Simulated crane placement (refer to Figure 6), including:
    - Varying wave conditions,
    - Two different "slings" and quick release mechanism,
    - Different toe details.

Figure 6 – Simulated Crane Placement in Model and Actual Placement

These test results confirmed the ability to place filter stone and armour units during typical swell conditions (including achieving the required placement density for the armour units), and also confirmed the stability of the resulting structure under the design wave conditions. The tests also led to refinements in the toe detail, and to the recommendation to use conservatively sized armour units (to reduce down slope settlement of the armour layer). Regardless, the ability to construct a suitable under layer in the field, and also to achieve good placement of the armour units, remained a significant concern due to the persistent swell wave conditions and the need for divers to verify placement under dangerous conditions.

Although a study was planned to address the second issue noted above (i.e. risk of armour unit breakage), this study was not completed, as the breakwater design concept was switched from concrete armour units to a berm design when a suitable source of quarried stone was confirmed.

Two berm concepts were considered, including:

- Single-class berm breakwater;
- Multi-class berm breakwater.

Baird developed the single class berm breakwater concept in the early 1980s, based on historical precedents and extensive physical model testing for projects in Iceland and Alaska. The design concept is based on the use of a wide, porous berm of armour stone to dissipate wave energy. This allows the use of smaller armour stone than a conventional design, with better utilization of the quarry, and provides an alternative to concrete armour units for exposed locations, with the potential for significant cost savings. Baird's most recent application of this concept was for the rehabilitation of a damaged breakwater in the Azores (refer to Scott et al, 2006), where a 30 m wide berm of 5 to 15 tonne stone was utilized to resist a design breaking wave of Hs = 7.5 m.

The multi-class berm breakwater was developed in Iceland, and has been widely used there over the past 20 years. This concept utilizes several narrow graded classes of armour stone, with the largest stone placed in the critical (most exposed) areas of the cross-section, thereby providing a lower value of $Hs/\Delta D_{n50}$ and increased stability, with the potential for more efficient use of the quarry. This concept has been recently applied to two breakwaters in Norway with design wave heights in the order of Hs = 7.5 m (refer to Sigurdarson et al, 2005).

An evaluation of the two-berm breakwater concepts was undertaken. The multi-class concept was identified as the preferred design concept, due to the fact that it was anticipated that this approach would achieve the required performance objectives (i.e. resist the extreme design event without significant stone motion or profile development) with a smaller volume of stone, without a significant increase in unit costs. Preliminary design development was undertaken based on published design guidance (such as PIANC, 2003), assuming three classes of armour stone (2-6 t, 6-12 t and 12-18 t). In addition, the design cross-section incorporated an interim armour layer to provide temporary protection against the persistent swell conditions and to meet the fast track project schedule (dredging and quay construction require the protection of the breakwater). The preliminary design cross-section is illustrated in Figure 7 (the dark area is the interim armour layer).

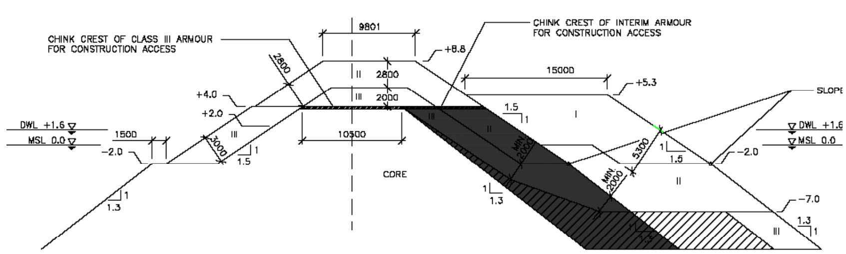

Figure 7 – Preliminary Design Cross-Section for Breakwater

PHYSICAL MODELING AND FINAL DESIGN OF BERM BREAKWATER

Two separate three-dimensional physical model investigation were undertaken to support final design development for the port and breakwater. Wave agitation and ship response tests were undertaken at a scale of 1:70 in a 20 x 30 m wave basin at the Canadian Hydraulics Centre. These tests were used to develop an estimate of downtime at the primary berth, and to refine the overall layout of the port (navigation simulations were also undertaken to support this process.

The breakwater stability tests were undertaken at a scale of 1:50 in a 24 x 30 m wave basin at HR Wallingford. The overall testing program included wave calibration tests, two tests of the interim cross-section and three tests of the full cross-section. Most of the tests were undertaken with the critical wave direction (East), but the final test was repeated with SE waves. Test conditions ranged from typical swell conditions (Hs = 2 m) up to extreme cyclone waves (Hs = 10.5 m), with depth-limited breaking waves at the breakwater. Figure 8 presents an overview of the model at HR Wallingford.

Figure 8 – Overview of 1:50 Scale Model of Breakwater and Port

The first test of the interim cross-section (crest at working level of +4 m MSL, with outer slope protected by two layers of 2-6 t armour stone) showed no damage at Hs = 2 m (average wave condition), with “tolerable damage” in most areas at Hs = 4 to 6 m. Severe damage to the interim armour layer, and overwash of the working platform, was noted along the outer 200 m of the breakwater due to wave focusing effects caused by a shoal. A second test with a more robust cross-section (6-12 t armour stone and working level of +6 m MSL) showed “tolerable damage” in this critical area.

The first test of the full cross-section was successful, with the model breakwater surviving exposure to extreme breaking wave conditions without significant profile development. However, initial stone motion was observed under moderate wave conditions (Hs = 4 m, an annual event), with continued exposure to waves resulting in down slope displacement of the 12-18 tonne armour stone from the primary berm (i.e. the initial stage of development of an S-shaped profile). This is illustrated in Figure 9 (the templates show the as-built profile).

Figure 9 – Profile Development Following First Test of Full Cross-Section

Although the overall performance of the breakwater in this first test of the full cross-section was deemed acceptable, the stone motion was considered to be undesirable for two reasons:

- Stone motion increase the risk of stone degradation and breakage;
- Loss of larger armour stone to lower slope represents a waste.

Two additional tests were undertaken with refined cross-sections, with various modifications made to reduce stone motion and profile development, including:

- Reduced slope on upper berm;
- Tighter placement of armour stone above water level;

In addition, various transition details were tested between the “root” of the breakwater (located in the shelter of Pointe Ehoala) and the full cross-section. The final test included detailed documentation of breakwater profile development throughout two design events, as well as extended test durations following the storm peak to assess residual stone motion. In addition, sensitivity tests were undertaken to assess the impact on stability of smaller (10-15 t) or larger (18-25 t) stone in the primary berm. Figure 10 presents profile measurements collected during the final test series; these results demonstrate the limited profile development that occurred in the final test.

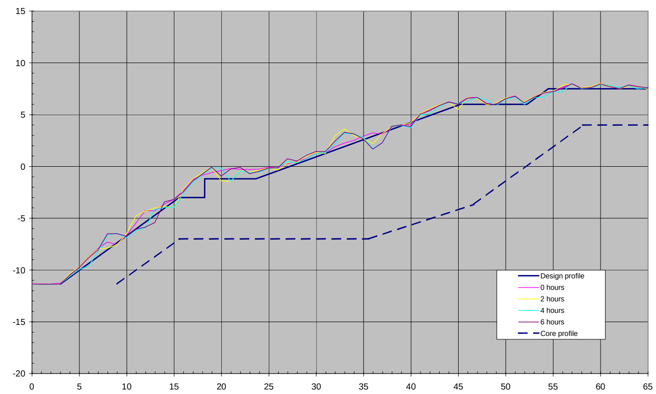

Figure 10 – Profile Measurements During Final Test of Full Cross-Section

Figure 11 illustrates the final design cross-section, as tested in the physical model (the dark area is the interim armour layer). Relative to the preliminary design (refer to Figure 7), the final design is more efficient, providing better performance (i.e. reduced stone motion and profile development) with a smaller total volume of stone, and a reduced proportion of the largest stone class.

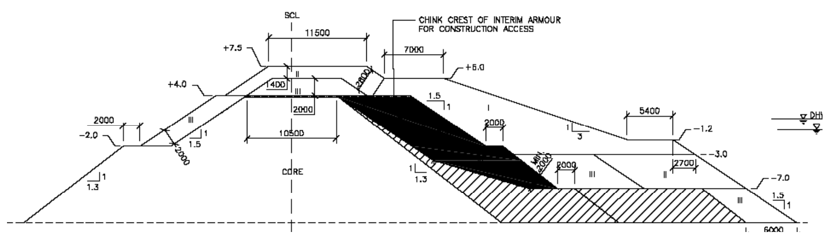

Figure 11 – Final Design Cross-Section for Breakwater

BIDDING AND CONSTRUCTION

The Port d’Ehoala project was bid using the World Bank’s two stage process. Five contractors were pre-qualified and submitted Initial Technical Proposals (ITPs) in June 2006. As expected, the five ITPs included varied construction approaches and equipment, and several different structural concepts for the quay structure. Following a detailed review of the ITPs by the design team, four were accepted and one was rejected.

Four final proposals (technical and financial) were submitted in August 2006. Following detailed evaluation by the project team, the contract was awarded to Daiho Corporation of Japan in November 2006. Their design concept for the quay consisted of interlocking steel pipe piles (with a two stage corrosion protection system) and a reinforced concrete coping wall.

Given the remote project location and limited local infrastructure, mobilization and startup was a major challenge/effort, and extended into the middle of 2007. Breakwater construction began in June 2007, with interim armour placed as the core was advanced. The full length (600 m) was reached in October. Completion of the full cross-section (i.e. berm, crest and rear slope armour) will proceed from the roundhead back. Land reclamation began in July, and dredging of the quay area began in August. As an aside, Daiho's geotechnical investigation (which was completed from a self-elevating platform as the breakwater was advanced) confirmed the assumed soil conditions presented in the bid documents (as interpreted from the earlier geophysical investigations), so no significant modifications to the quay design were required. Figure 12 provides an overview of progress at the site as of September 2007.

Figure 12 – Overview of Construction – September 2007

CLOSURE

Construction of the mine and port are proceeding on schedule, with the first shipment of ilmenite expected to leave Port d'Ehoala in late 2008. The project has proceeded very smoothly, demonstrating a success story in private/public partnerships. The completed port will meet QMM's needs, and will also provide a catalyst for economic development in the region. The successful implementation of this large, complex project is in large part due to the exceptional quality of the QMM team, and Rio Tinto's corporate commitment to the people and environment of Madagascar.

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