ABSTRACT

This article reviews the design criteria and design development from option study to detailed design of an offshore island and causeway development off the Eastern Province of Saudi Arabia. The Manifa project, which is currently under construction, consists of 21 km of main causeways and 21 km of lateral causeways connecting to the 27 drilling islands and will support oil production of 900,000 barrels/day. The islands are each 340 m x 265 m (or 9 ha, about the size of 13 football pitches). The rock armour revetments are approximately 120 km in extent and the works require the dredging and land reclamation of approximately 37 million m³ and the placement of 10 million tonnes of rock. In order to fulfil environmental requirements openings have been introduced to the causeway which are bridged by 4 km of bridges including one of 2.4 km.

Three concept options are briefly discussed:
- Scheme A: the construction of 27 drilling islands which are linked by causeways to the land;
- Scheme B: the construction of 27 drilling islands which are grouped in isolated clusters by means of causeways. Access to each cluster would be from the sea;
- Scheme C: the construction of two water injection islands and associated minor causeway together with dredging works for offshore platforms.

The development of the design parameters for the project and their implementation in detailed design as well as some critical practical aspects such as the quality and availability of materials – primarily rock and sand – are then examined.

Parameters discussed are:
- Design water level;
- Design waves and return period;
- Overtopping;
- Wave transmission;
- Earthquake loads;
- Water circulation;
- Coastal morphology;
- Cross-section design;
- Armour stability and model testing;
- Rock quality and availability; and
- Geotechnical aspects.

Of particular interest is the section discussing compensatory measures for the use of marginal quality rock given restricted availability of good quality rock in the Arabian Gulf at the present time.

Client is Saudi Aramco and Jacobs Engineering were the concept designer and the Client’s Technical Advisor. The Contractor is Jan De Nul nv. Contract is a Lump Sum Turn Key (LSTK) with the Contractor responsible for detailed design.

The authors acknowledge permission of Saudi Aramco to publish this paper and Its copyright material contained herein. Further original data sources are acknowledged as below: Figures 1, 2, 3, 6, 7 by DHI; Figures 4, 5 by Jan De Nul; Figures 8, 12 by Saudi Aramco; and Figure 9 by Jacobs UK Ltd and Figure 10 by David Close.

INTRODUCTION

With increasing world demand for oil, Saudi Aramco contracted to bring its mothballed Manifa heavy oil field back into production with an ultimate production capacity of 900,000 barrels of oil per day.
The Manifa oil field is situated just offshore of the Eastern Province of Saudi Arabia in the shallow coastal waters of the western Arabian Gulf that generally have depths of less than 5 metres. The extensive shoals would require either extensive dredging to create access channels for offshore jackets or the creation of drilling islands and access causeways for road access for land-based drilling rigs. Saudi Aramco commissioned a fast track feasibility study followed immediately by procurement of a Lump Sum Turn Key (LSTK) contract for the design and construction of the preferred option.

Construction commenced in early 2007 of 21 km of main causeways and 21 km of lateral causeways connecting to the 27 drilling islands. The islands are each 340 m x 265 m (or 9 ha, about the size of 13 football pitches). The rock armour revetments are approximately 120 km in extent and the works require the dredging and land reclamation of approximately 37 million m³ and the placement of 10 million tonnes of rock. To fulfil environmental requirements, openings have been introduced to the causeways that are bridged by 4 km of bridges, including one of 2.4 km length.

**CONCEPT DESIGN**

Saudi Aramco undertook its preliminary reservoir engineering in 2005, fixed the numbers and locations of the islands required for drilling and water injection and made a preliminary assessment of causeway alignments and widths.

The concept study evaluated three basic schemes being:

- **Scheme A**: The construction of 27 drilling islands which are linked by causeways to the land.
- **Scheme B**: The construction of 27 drilling islands which are linked by causeways to the land.

![Figure 1. Mike 21 Mathematical Model of Manifa Causeways and Islands looking from the Southeast.](image1)

![Figure 2. Scheme A - Easterly Waves](image2)

![Figure 3. Scheme B - Easterly Waves](image3)
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islands which are grouped in isolated clusters by means of causeways. Access to each cluster would be from the sea.  
• Scheme C: The construction of two water injection islands and associated minor causeway together with dredging works for offshore platforms. This scheme would have full offshore production.

Saudi Aramco evaluated whole life costs of the three schemes (marine and civil engineering, and electrical, communications and pipelines which were the subject of separate parallel studies) and assessed programme estimates and construction risks in coming to its conclusion. Scheme A was selected for the works.

DESIGN BASIS

Given the fast track nature of the works, Saudi Aramco had already commissioned the King Fahd University of Petroleum and Minerals (KFUPM) to undertake an Environmental Impact Assessment. As part of the EIA work, DHI had been contracted by KFUPM to investigate water circulation and had already established an offshore model for the Ras Tanajib area (see Figure 1). DHI had also been nominated as a sub-consultant for the concept design study.

The hydraulic study was tasked with providing:
• Design water level data for the perimeter structures of the islands and the causeways including sea level rise over the lifetime of the structure;
• Design waves for perimeter structures of the islands and the causeway;
• Overtopping for design conditions;
• Operational wave and current conditions for the planning of dredging operations;
• Input to downtime statistics in terms of wave statistics for three berthing locations.

Hydraulic parameters are considered in more detail below.

Design water Level

The existing water level prediction for the Manifa project was not considered to be sufficiently accurate. DHI undertook a study using a combination of water level measurements recorded at Saudi Aramco’s Ras Tanajib Pier (1985-2005) and the PERGOS database. This database includes numerical hindcast model results of more than one hundred historical storms over the period 1983-2002. The recommended values for extreme tides from the study were:
• MSL is 1.0 m above LAT;
• HAT is 1.8 m above LAT;
• 100-year storm water level is 2.2 m above LAT.

An average sea level rise of 5 mm per year has been assumed resulting in a water level increase of 0.25 m over the next 50 years. This assumption was based on the contemporary Intergovernmental Panel on Climate Change (IPCC) predictions. The end of life (50 years) prediction of the 100 year storm water level was therefore assumed to be 2.45 m above LAT, rounded up to 2.5 m above LAT.

Design waves

For islands exposed to the most severe 100-year easterly direction, the maximum significant wave height (see Figures 2 and 3) was 2.8, with a peak period of 9.1 s. The variation in significant wave height was from 2.4 m to 2.8 m. This situation remained the same for both Schemes A and B, but the loss of the main causeway in Scheme B caused a significant change to design wave conditions in the westerly islands which is apparent when comparing Figures 2 and 3. During the course of detailed design, much more detailed investigation of the wave climate was undertaken as indicated in Figures 4 and 5.

Overtopping

An understanding of overtopping is critical to defining the crest level. The quantity of permissible overtopping must first be defined. At the concept stage, a figure of 2t/m/s was selected from a consideration of published overtopping damage1 having due regard to the nature of the facilities on the causeway. There will be occasional small facilities on the structure for consideration

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of overtopping will be electrical supply cables, access roads and pipelines. During the detailed design stage, the overtopping limit was reduced to 1 l/s/m.

The wave overtopping criterion is traditionally presented as the volume of water per second per metre of revetment which presents the non-coastal engineer with a difficulty in comprehension as the overtopping figure seems so small. It has to be realised that the overtopping is caused by a few wave events during a storm, typically less than 2% of waves, so that the actual volumes within a single wave event can be considerable. As an illustration, assume that 1 l/s/m is used as the criterion; the total volume per metre during a 3-hour storm would be 3600 l/m. Most of this volume will be carried by, say, the largest ten waves, which then means that a volume of approximately 3.6 m³ of water passes over a 10 m long revetment section during such individual events.

Figure 6 was derived during the concept study as an aid to defining the necessary freeboard to fix the rock armour crest level.

Overtopping is a stochastic and highly varying parameter that makes it difficult to produce empirical relationships that will yield accurate results. Different formulations can produce results with large differences. Overtopping was therefore studied by physical modelling during the design process in order to set the crest elevation.

**Wave transmission**

From early discussions with dredging and land reclamation contractors active in the region, it was envisaged that the cross-section of the causeways and islands would be sand fill with or without quarry run shoulders. Together with the rock armour, this form of construction represents a very porous structure and a major consideration in setting fill levels was therefore the degree of damping of waves by the structure and the consequent elevation of the crest of transmitted waves or standing water levels within the causeway or island body.

Empirical equations due to Barends’ exist for the definition of water levels within land reclamations but physical modelling was undertaken during the detailed design phase to define fill levels as well as armour crest elevations.

**Earthquake**

The Eastern Province of Saudi Arabia is not seismically active but, after the recent Iranian earthquake felt in the United Arab Emirates some hundreds of kilometres further south, the Client imposed a cautious 5% g earthquake requirement for this structure.

**ENVIRONMENTAL CONSIDERATIONS**

**Water circulation**

Scheme A represented the most potentially damaging environmental proposal as it would have closed off water circulation if constructed as a solid structure. It was therefore always envisaged that provision for the maintenance of water circulation by the creation of openings would have a high priority.

King Fahd University of Petroleum and Minerals (KFUPM) undertook the
environmental assessment\(^3\) with DHI modelling water circulations under KFUPM’s direction. A variety of scenarios were investigated ranging from the provision of 20% openings throughout the length of the structure, through to combinations of small openings throughout the length of the causeway and a larger opening at the root of the main causeway. The optimum Scheme A causeway layout (see Figure 7) has a 2.4 km long bridge near the land connection of the main causeway from the -3mCD contour to the nearest drill island and openings in the main causeway in the form of short bridges (150-m-long each) and culverts (50-m-long each).

The time it takes for 50% of the water in the Manifa-Tanajib Bay system (or MTBS, the enclosed bays in the modelled region shown in Figure 7) to be exchanged with the Gulf waters is 17 days in the existing situation. That would increase to about 71 days if a solid causeway were created. Introducing large-scale openings amounting to 20% of the length and distributed throughout the causeway length would reduce the T50% to 20 days. The combination of a long bridge at the southeast and 5% openings through the main causeway results in a residence time of 15 days, which represents an improvement in the current situation.

Overall, the potential hydrodynamic alterations are expected to result in tidal pumping which will generally benefit the water exchange efficiency in the MTBS and the coastal areas south of the causeway onshore approach. Tracer concentration simulation and salinity modelling revealed that water conditions will improve in these affected areas due to a higher rate of water renewal resulting from the intensified flow regime.

In connection with the effects on local hydrodynamics, changes in basic water quality conditions (water temperature, salinity and dissolved oxygen) will not be of serious concern. The average increments in water quality parameters at the local and regional scales are generally negligible and perceived to within the tolerance limits of marine organisms. All expected increases are also within the ranges of natural variability in the MTBS and the Gulf area, in general.

**Coastal morphology**

The effect on coastal morphology caused by the causeway will be very small and undetectable from all other natural changes except very locally where the causeways are connected to the shore. The coastal water along the entire stretch is very small so that larger waves which could move significant quantities of sand cannot come close to the coastline in the existing situation, thus future sheltering of the coastline by the planned causeway will not have an effect.

Close to the two shore connections some local accumulation of sand and fines will develop on the north side of the structures due to the dominant winds from northerly directions. At the southern shore connection a similar pattern will develop on the south facing side of the structure due to the rare but more powerful southeasterly winds. These accumulations will be the result of local generated waves in the nearshore zone and this pattern will develop at all sites on the coast where an obstacle across the shoreline is made. Such small changes will not have any impact on the quality of the existing coast nor will they disrupt any larger sediment circulation cells.

There will be localised areas of water stagnation behind the main causeway, especially in between the projecting branches (the lateral causeways leading to the islands), which may lead to increased siltation. The same sedimentation effect and the resulting sediment accumulation are also expected to increase the extent of finer substrates within the MTBS in the medium term.

**CROSS-SECTION DESIGN**

**General considerations**

An outline design of the causeway (see Figure 8), which was to be developed and optimised, was presented to Jacobs at the commencement of the study. This outline design envisaged fill to 2m above MSL (+3mCD) and a rock armour revetment with its crest at 5m above MSL (+6mCD). The outline design envisaged the revetment to comprise 1-3 tonne rock armour.

In connection with the effects on local hydrodynamics, changes in basic water quality conditions (water temperature, salinity and dissolved oxygen) will not be of serious concern. The average increments in water quality parameters at the local and regional scales are generally negligible and perceived to within the tolerance limits of marine organisms. All expected increases are also within the ranges of natural variability in the MTBS and the Gulf area, in general.

Crest freeboard as function of slope for 2 l/s/m - NN equation

![Figure 6. Crest Freeboard as a Function of Slope under Overtopping Discharge of 2lt/m/s.](image)
Owing to tight time constraints, the concept study wave modelling derived wave heights for island groups rather than individual islands. It did, however, derive wave heights from different directions. It was therefore possible to design rock armour for exposed and sheltered sides of islands and causeways. Concept design armour sizes were derived from the well-known van der Meer equation and cross-checked using the older Hudson formula.

A concern of the concept study, and one which the Client had recognised from his own studies, was the availability of rock for the works given the very active state of marine construction within the Arabian Gulf. Enquiries with major Contractors involved in these ongoing, very large, prestige projects confirmed that rock supply was likely to be a major concern for tenderers for the Manifa contract.

This concern affected the manner in which the concept study was conducted and the concept designs were therefore optimised on 1-3 tonne rock with slopes varied to suit instead of using steeper slopes with heavier rock. It was considered that this grade of rock would be more readily available.

The concept design (see Figure 9) for the revetment on the exposed side was standardised on an average slope of 1 in 2.25 with rock armour of 1-3 tonnes. The crest of the causeway was set at +5mCD (4m above MSL and 2.5m above the 1 in 100 year storm water level) and the crest of the armour set from overtopping considerations at +5.5mCD (3 m above the 1 in 100 year storm water level).

Figure 9 shows a cross-section of the main causeway with the pipelines set on the sheltered side and electrical cables fixed on trays in a pre-cast U-shaped concrete channel. The concrete channel is set above the level of the rock armour crest to provide shelter to the cables from overtopping waves. The height of the seaward side of the channel has been determined by physical modelling to maintain the set overtopping limit.
Armour stability

Armour stability of many sections, armour sizes and exposures were investigated during detailed design by physical modelling in the small scale flume at the University of Ghent, and the crest and cable channel design was similarly investigated in the university’s large-scale flume (see Figure 10). Owing to the strategic value of the infrastructure being designed, the design storm return period was defined in collaboration with the Client at 1 in 100 years. The majority of the offshore installations in the region have also been designed for a 100 year return period event. During detailed design, consideration was given to increasing the return period to 1000 years but a cost / damage assessment confirmed the lifetime cost effectiveness of the 1 in 100 year specification.

Owing to the relative conservatism of the concept design which limited damage in the design storm to a maximum damage number of $S = 2$ (equating to 0.5% of stones displaced from the active zone) over 3000 waves (a storm duration of about 7.5 hours in prototype), it was not considered necessary to increase the return period of the design storm. However, a close view was kept on the outcome of the flume tests that used waves up to a return period of approximately 1 in 1000 years. While damage obviously increased markedly at return periods above that of the design 1 in 100 year storm, no breaches of the revetment appeared likely even under the 1 in 1000 year storm. The estimate of the cost of damage repairs made by the designer (who was the Contractor) also justified the selection of the lower return period.

Rock quality and availability

One of the main concerns of the Client before and during procurement was the availability of rock armour of sufficient quality and in sufficient quantities for the works given that 10,000 tonnes per day of armour rock would be required to meet his programme (Figure 11). The tender was written around “good quality” rock but with the flexibility, except for limited areas around openings, given to contractors to use “marginal quality” rock provided that provision was made for degradation in accordance with recommendations from the Manual on the Use of Rock in Coastal and Shoreline Engineering with the exception of the option of increased maintenance. That is, increased rock degradation of “marginal quality” rock could be compensated for by:

- Over-dimensioning of armourstone;
- Gentler seaward design slopes and increased volumes of material;
- Combinations of the above.

The shortcomings of the Manual on the Use of Rock in Coastal and Shoreline Engineering were apparent once the contract started owing to the lack of availability of (Queen Mary & Westfield College) abrasion mill apparatus used to define the mill abrasion index on which the recommendations were based. Accordingly, the updated test criteria were sought from the authors. The rock degradation model referred to in the Manual on the Use of Rock in Coastal and Shoreline Engineering has been updated and is now based on the more readily available micro-Deval test. These new conditions and criteria were published shortly after contract award in the updated 2007 Rock Manual.

Marginal quality rock meeting the contract specifications based on the Manual on the Use of Rock in Coastal and Shoreline Engineering is available within a relatively short distance inland of the site. A degradation allowance was made based on curves derived from the rock degradation model described by Latham et al. (see Figure 12) using quarry-derived material parameters.
Geotextile

The revetments of the causeway and island structures were designed at concept using geotextile below underlayer and armourstone layers. This system was adopted by the Contractor in his detailed design and trials of the proposed underlayer were undertaken before acceptance for incorporation into the works. Full-scale trial embankments were constructed on land and were subsequently (and carefully) dismantled to prove the sufficiency of the proposed geotextile.

Geotechnical

A comprehensive offshore geotechnical and geophysical investigation programme was undertaken by the Client prior to tendering in order to assist the bidders in evaluating the availability and suitability of locally won soil for reclamation. Minimum criteria were set pertaining to chemical properties (e.g. carbonate and organic content), physical properties (e.g. fines content, gradation, specific gravity and bulk unit weight) and in-situ properties (e.g. percentage of maximum dry density). Placement and compaction criteria were specified for fill placed above mean sea level (i.e. in the dry). Higher quality fill was specified behind or adjacent to structures, within 1000 mm below roads, pipe and cable zones and bridge approaches. Flexibility has been given to the Contractor on the placement and compaction methods to achieve the set performance criteria that included settlement limits at 5, 25 and 50 years.

The design provides compensation for any remaining primary consolidation, elastic compression and any future secondary settlement/consolidation. Long-term stress-strain behaviour has been studied to evaluate the residual (creep) settlement.

In order to address concerns about the suitability of locally won sand because of its high carbonate content, crushability of two sand samples was tested by normal Proctor compaction and dynamic oedometric loading. The material did not exhibit major crushability at the above mentioned stress levels and settlements were predicted to be within the performance specification for the works.

CONCLUSIONS

Initial high level coastal modelling has shown that the impacts of the proposed causeway on the existing environment can be minimised or, in the case of the selected option, even improved.

The initial concept design was based on limitations to the rock size that would be

Figure 10. The sequence of photos shows the progression of an overtopping 1 in 100 year wave (Hs = 2.75 m, Tp = 9.0s) towards the cable channel on the seaward crest of the main causeway. The large wave flume gave a 1/15 scale model. Modelling took place in the large scale wave flume at the University of Ghent, Belgium.
available in the region. The selection of size was made following discussions with local and international marine contractors who were active in the region. The contract documents were developed to enable the appointed LSTK Contractor to use sub-optimal quality armour stone, provided compensatory measures were taken.

The design storm was initially set at 1 in 100 years in discussion with the Client. A cost / benefit assessment of upgrading the design to accommodate a 1 in 1000 years storm undertaken by the LSTK Contractor demonstrated the cost advantage of the specified 1 in 100 years storm.

Detailed numerical modelling by the designer enabled large economies to be made. Physical modelling enabled further refinement to the cross-section and slope design to be made and gave confidence in the performance and stability of the structure in storms of and exceeding the design 1 in 100 years.

Figure 11. The availability of rock armour of sufficient quality and quantity was a main concern of the Client. Shown here, a rock drop test on large armourstone at the quarry at Nuariyah.

Figure 12. Predicted Rock Degradation Loss at End of Structure Life (50 Years).

REFERENCES


