Design of marine facilities

EG LNG train 1 project – Bioko Island, Equatorial Guinea

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In January 2005 Besix undertook to design and build a new LNG marine facility located on Bioko Island, for client Bechtel. The scope included an LNG loading platform and berth, a marine operations platform, a small boat dock, and a trestle connecting the loading platform to a cable stay bridge carrying an onshore piperack. In addition the scope included all topside works such as piping, equipment, utilities and support buildings located on the two platforms.

The project was completed successfully in January 2007 with the first LNG berthing at the facility on 24 May 2007.

Figure 1. Bioko Island Bechtel LNG marine facility.

Basic design requirements

The jetty was designed to accommodate vessels of 160,000 m^3 capacity and it must be able to withstand 100-year storm events with maximum wave heights of up to 5.6 metres.

Hmax is assumed to be equal to 1.85 Hs.

TABLE 1: JETTY DESIGN REQUIREMENTS

Design Condition	Type	Return Period	Sign Wave Height Hs	Period, T
Operational	swell	1:10	1.9	15
Extreme	squall	1:100	3.0	6

Design was based on American Petroleum Institute (API) and ACI standards. The design life of the jetty is 20 years. In terms of steel pile design this translated as 6 mm corrosion allowance within the splash zone.

Reinforced concrete structures were designed for a crack width of 0.2 mm.

Post-tensioned single-T beams were designed as class two members with a requirement of zero tensile stresses under sustained dead and superimposed dead loads. Under dead plus live loads tensile stresses were permitted as long as the tensile capacity of the concrete was not exceeded, hence ensuring that the sections remained un-cracked under all loading conditions.

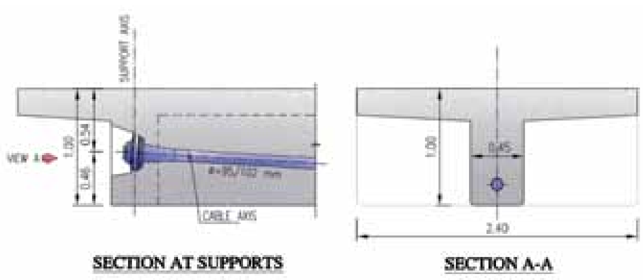

Figure 2. Post-tensioned single-T beams.

Seismic design

Bioko is located in an area of high volcanic activity hence the

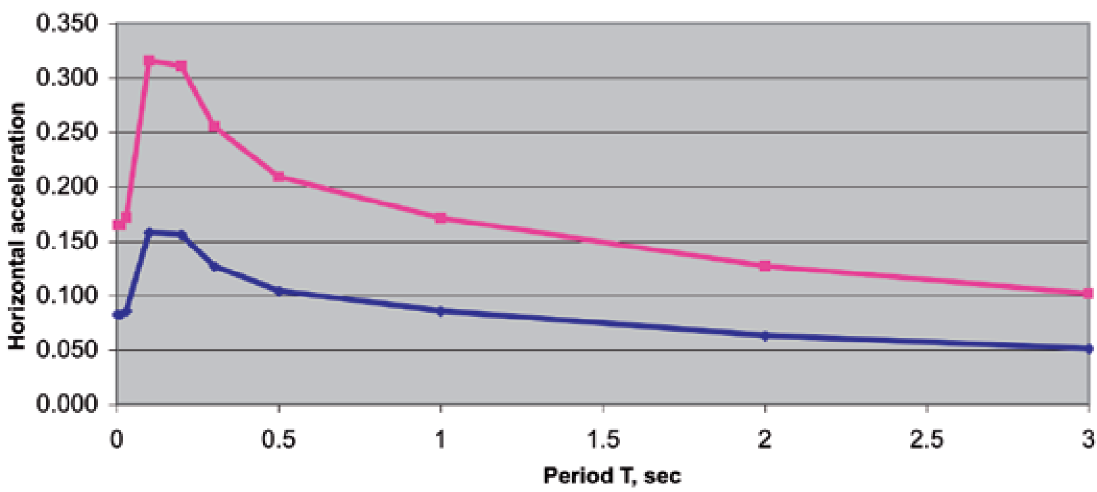

Figure 3. Strength level (SLE) and Ductility level (DLE) seismic spectra.

need to allow for seismic loading in the design. The analysis is based on the precondition that the number of modes considered in the modal analysis have to satisfy the requirement that at least 90 per cent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

Two levels of earthquake were considered, namely the Strength level (SLE) and Ductility level (DLE).

SLE requires that the structure should remain fully operational under earthquake shaking which has a reasonable likelihood of not been exceeded during the design life of the structure. The SLE earthquake was checked against both ULS and SLS combinations against operational cases. Detailed calculations confirmed that SLE earthquake is not critical and produces less stresses than the normal operating conditions.

Under DLE analysis the structure is expected to suffer some level of structural damage but has to have adequate reserve capacity to ensure that collapse does not take place. DLE is a rare case event with very low probability of occurrence. The DLE earthquake was checked at ULS against loads from the operational conditions. The DLE earthquake was found to have no effect on the structural capacity of the jetty at ULS although it causes some uplift on some of the piles at SLS. The embedded length was checked and found to have sufficient capacity even with a severe factor of 2.0 on pile skin friction. Deflections were also more critical but within acceptable limits regarding structural stability.

Sub-structure

The jetty is founded on 1,220mm diameter tubular steel piles raking at a maximum of 1:4. The piles were driven to the following penetration lengths:

• Trestle	20.00 m (21.5 m for section interconnecting MOP and LP)
• MOP	25.75 m
• Loading Platform	34.00 m
• Berthing Dolphins (East)	34 m (West) 38.00 m
• Mooring Dolphins (East)	27.00 m (West) 34.00 m

Crossheads consisted of precast concrete boxes which were supported on the piles temporarily by means of steel embedded sections. The reinforcement cages of the crossheads were prefabricated onshore and were lifted in position using the jack-up platform. The precast sections were designed to act in composite action with the in situ infill in order to optimise both the size and weight of the sections and rebar. A staged design approach took into account the build-up of stresses during the various temporary and permanent stages.

Super-structure

As with all oil-and-gas related projects timely completion of the job was of paramount importance. In order to ensure that all works could be carried out in a timely manner the designers had frequent work shops with the various project teams to ensure that design was streamlined to meet the capabilities of the equipment and marine plant available to the site.

Concrete supply

A second consideration was the remoteness of the site and the extremely difficult process of delivering concrete. This was done by loading ready-mixed concrete to 9 m^3 tracks mounted on barges which were then towed to the jetty. The trip took two hours and thirty minutes and required very tight control at the concrete plant to ensure that all mix constituents were in accordance with design and specifications. For this reason it was decided at a very early stage in design to replace as much as possible any in situ concrete with precast elements.

Figure 4. The precast, post-tensioned 2.4 m wide beams were arranged in such a way as to accommodate distinct loadings and services.

Trestle construction

In order to avoid any in situ concrete connecting them to the crossheads (headstocks) the beams of the trestle were designed as simply supported over 23.5 m spans. Furthermore, the trestle was sub-divided into three distinct areas which led to the design of three individual precast, post-tensioned 2.4 m wide beams, arranged in such a way as to accommodate distinct loadings and services:

- Beam 1 – Walkway, parapets and cable trays
- Beam 2 – Two LNG 30" pipes
- Beam 3 – Remaining miscellaneous piping on two levels

The beams are supported on two steel bearings at each end. First they were lifted in position and adjusted to the correct elevation using temporary shim blocks. Finally the bearings were grouted in box-outs left in the crossheads. As soon as the grout gained sufficient capacity the beams became fully functional without the need of any further in situ concreting or other associated works. As a matter of fact all 63 beams were placed within the space of 10 cumulative days with a rate of up to nine beams per day, saving considerable time to the project.

Marine operations platform

The same beams were also used for the MOP. Since the MOP was designed to accommodate three different buildings with the difference that the sections had to be interconnected together by means of in situ concrete stitching in order to avoid differential deflections.

Figure 5. The sections of the MOP were interconnected together by means of in situ concrete stitching.

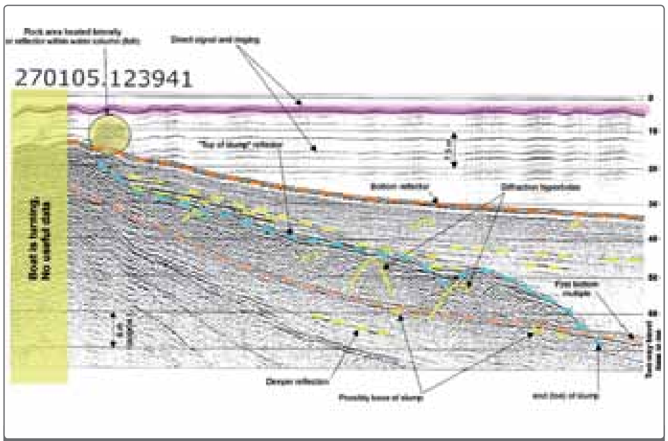

Figure 6. A side scan of the soil was performed to help assess mitigation measures in regards to the foundation.

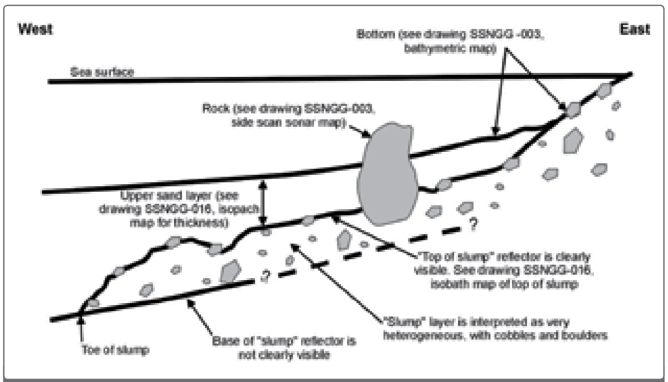

Figure 7. Additional analysis of soil revealed very large boulders present.

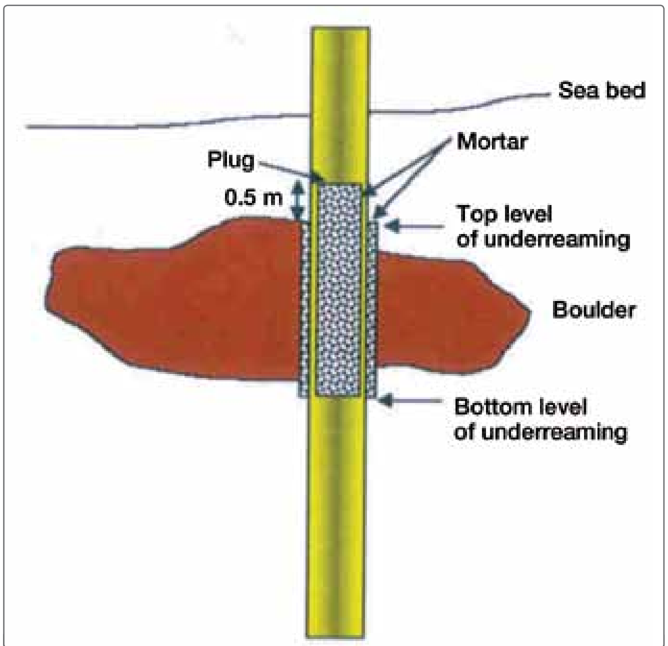

Figure 8. Under-reaming is necessary for penetrating large boulders.

Installation of the steel open-end piles

Despite the extremely extensive soil investigation reports which were available in tender stage, it was still very difficult for the design and build contractor to predict with sufficient accuracy the foundations of the jetty, consisting of 200 driven piles. The reason for this is the fact that between the +/- 20 boreholes available, only one was detecting a 'basalt layer' with a thickness of +/- 5 m at a depth of 5 m.

The piles at this location cannot be terminated on top of this relatively shallow 'basalt layer' because of their very limited tensile capacity. Instead they had to be drilled through that hard layer by an under reamer, and then driven again to the required depth.

Since this operation is very expensive and very time consuming, it was important to predict how many piles would be affected by this problem. A better understanding of the extent of that rock layer was required to better mitigate the associated risks.

Risk mitigation consisted of the following steps:

1. Accurate risk event description
2. Mitigation strategy and action plan
3. Estimation of the relative cost of the mitigation
4. Estimation of the probability of success if said action is implemented

In addition to the cost of mobilising an under-reamer, there is also the risk that under-reaming will not be possible if the pile toe is damaged during driving. And of course, there was also the commercial dilemma of who would bear the additional cost for an unknown number of these operations.

Additional soil investigations were performed to help assess these mitigation measures. These consisted of side scanning to detect potential outcrops of rock, geophysical bottom profiles, and additional borings.

Upon analysing the additional soil investigation data it became clear that there were no 'layers' of basalt, but that instead very large boulders were present; rip-rap in a 'slump' below sea bottom as a result of volcanic eruptions in the past.

The risk in encountering boulders

Under-reaming of large to very large 'boulders' is needed to penetrate through these rock blocks.

These blocks can be encountered over the full surface of the slump. Unfortunately, only the extreme east dolphin (four piles) was not in the slump location. This means that for 196 out of 200 piles there was a risk of encountering boulders.

Finally, it was agreed with the client (Bechtel) that Besix should take the risk of under-reaming up to 10 per cent of the piles. If this percentage was exceeded, the balance would be financially compensated by Bechtel.

Risk of pile tip damage

Under-reaming becomes impossible when the pile tip is damaged. This means that the pile has to be driven hard enough to reach the potential boulder, but soft enough to avoid damaging the pile tip.

Besix developed a methodology to reduce significantly the risk of damaging the pile tip, by modelling the pile behaviour during driving using dynamic pile driving software. The potential boulder is also modelled in the software. The experience, which was built up during driving in similar conditions, allows for defining with accurate precision the conditions when damage to the pile tip is initiated. This allows

Figure 9. Example of a damaged pile tip.

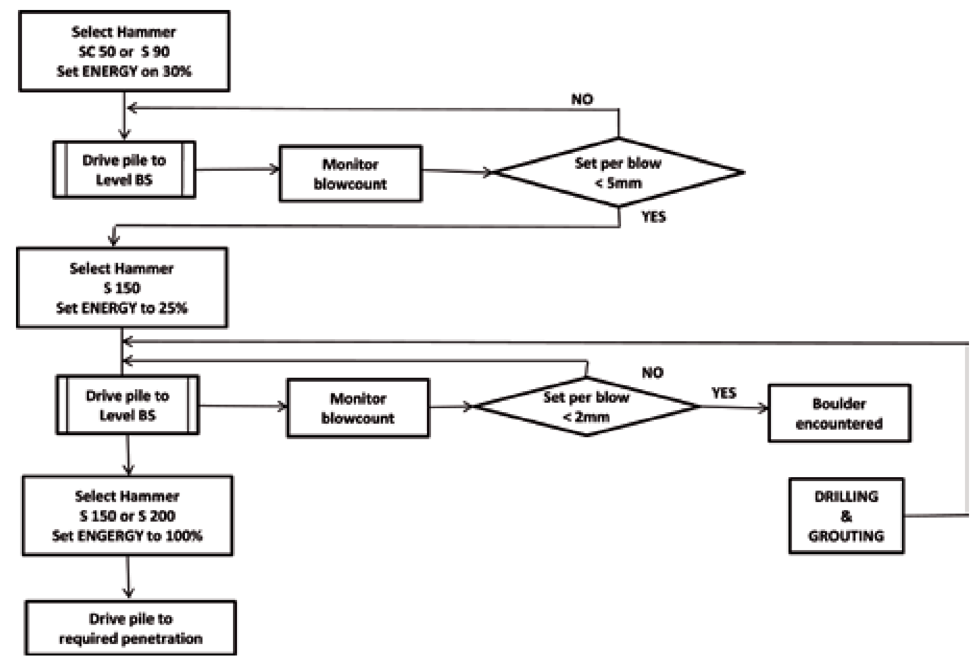

Figure 10. Pile driving procedure for negotiating boulders.

defining for each pile (diameter, wall thickness) the hammer type and the energy setting of the hammer to minimise the risk of damaging the pile tip.

Pile driving procedure

After finalising the design of the piles and the driving criteria to guarantee that the piles have the required bearing capacity at the design depth, a procedure has to be put in place to give enough comfort to all parties that this depth will be reached despite the presence of the boulders and despite the fact that the position of the boulders is unknown. What is known from the geophysical investigations is the extent of the slump and the bottom level of the slump (BS).

After performing a large number of driving simulations, a comprehensive installation procedure as given below was put in place: Piling was started with a light hammer to reduce the bending in the raking piles when the stick-out is still large, then a heavier hammer was used with a reduced energy setting until the BS is reached. Finally the pile was driven to the required depth with a heavier hammer and a 100 per cent energy setting.

Conclusions

For Besix this was a very difficult project which combined the usual problems of a remote site with some very unpredictable soil conditions. The presence of 'boulders' forced the design teams to look for innovative new ideas on how to ensure the timely completion of the project within the allocated budget. During execution the site teams had virtually 24 hour access to the designers in resolving any technical issues associated with piling and ensuring continuous operations. Thorough technical understanding of the installation procedures by the site teams and genuine team spirit between Besix and Bechtel design engineers ensured that all piling operations were completed successfully.

The simplicity of the deck design was another important component. The aim from the start of the project was to 'keep it simple' and to ensure that all potential problems were anticipated during the design phase. The remoteness not only of the site but also of Bioko Island itself meant that any problems that required outside support would have been too costly and would have taken too long to resolve. Again, good coordination between the design and site teams was the key to success beginning at the drawing board and ending at the time that the jetty was handed over to the client.

ABOUT THE AUTHORS AND COMPANY

Luc Maertens, Civil Engineer, is Technical Director at Besix. Mr Maertens has more than 35 years experience in marine structure design working on such projects as: Quay walls in Belgium, Tunisia, Congo, UAE, Mauritius, and France; Sea Locks in Zeebrugge; Near shore Terminals in India, Libya, Egypt, Equatorial Guinea, UK, and Egypt. He is a Professor at the KUL, a Member of Permanent Committee IABSE, and a Member of the Royal Academy of Oversea Sciences (BE).

Michael Paschalis, MSc Structural Engineering and Foundations, is currently Director of Besix' Engineering section in Dubai. Mr Paschalis has accrued 10 years' experience in structural design of projects in UK, Hong Kong and Malaysia. As part of Besix he was the project manager for the LNG Terminal project in Dabhol (India), Gas Terminal Wafa (Libya), and the LNG Jetty in Bioko (Equatorial Guinea). He performed the tender design for jetties in Angola, Australia Oman, and UAE. He is a Chartered member of the Institution of Structural Engineers London (MIStructE, CEng).

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