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System for automated design of offshore rock berms

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\textbf{ABSTRACT:} A system for automated design of offshore rock berms is described. The main idea is to extract 2D sections on the basis of the actual 3D geometry (seabed topography and berm geometry). These sections are then processed, first with a bespoke Limit Equilibrium procedure and next via Finite Element Limit Analysis. The end result is a rapid method for automated design of offshore rock berms. In the present paper the seabed is a soft clay, but the methodology is in principle applicable to any material whose strength characteristics is described reasonably with the standard Mohr-Coulomb failure criterion.

\textbf{Keywords:} Finite element limit analysis, limit equilibrium, stability analysis, rock berms, offshore geotechnics.

1 \textbf{INTRODUCTION}

Iceberg plough marks from the last glacial period are a common feature of the seabed of the Norwegian Sea. The plough marks are in the form of long canyons some 2 to 10 m deep. An example is shown in Figure 1. Iceberg plough marks are a substantial challenge in the design of offshore pipelines. In particular, the question is whether the pipe line will be able to span a given plough mark freely within suffering excessive strains. If not, intervention in the form of a rock berm is necessary. Rock berms are embankment type structures constructed on the seabed on which the pipe line is supported to traverse the plough mark. The total volume of typical rock berms may range from a few hundred to tens of thousands of cubic metres. An example of a rock berm traversing a plough mark is shown in Figure 2.

The design of offshore rock berms is complicated by the fact that the seabed sediments often are very soft, typically with an undrained shear strength of a 1-2 kPa at the surface and increasing 1-2 kPa/m with depth. This often renders berms such as the one shown in Figure 2 unstable. To stabilize a given berm, so-called counterfill may be added. These are additional berm-type structures extending out from the main berm. An example is shown in Figure 3.

The design of rock berms thus comprises an iterative sequence of stability analyses with modification of the counterfill in each iteration to obtain the smallest possible final volume.

Although rock berms are obvious 3D structures, their design has traditionally been carried out by considering an appropriate selection of 2D sections. This approach is conservative (provided the critical sections are identified), but has advantages in terms of computational requirements, leading, generally speaking, to less total design time.

Also traditionally, rock berm design has been carried out in a manual, sequential manner with each rock berm along a given route being designed by first extracting a number of sections and then iteratively modifying the counterfill to ensure overall stability (Eiksund et al. 2008).
A typical berm requires consideration of some 10-20 sections. Assuming that 10 iterations are required to obtain a satisfactory design, some 100-200 2D stability analyses are required. Assuming further that a given pipeline route comprises some 50-100 berms, the total number of analyses is of order 5,000-20,000. Assuming 1 min of computational time per analysis puts the total computational time in the range of a few days to a few weeks. This time, however, would typically be eclipsed by the time required for setting up models, decision making in each design iteration, etc. As such, in the best case, the design of a complete pipeline route easily takes several months. Finally, if changes are made to the pipeline route, the entire process needs to be repeated.

In the following, a strategy for automated design of rock berms is described. The strategy is roughly equivalent to the manual one. First, relevant 2D sections are extracted based on an analysis of the topography. A typical section is shown in Figure 4. Stability analyses for each section are then conducted using a bespoke Limit Equilibrium (LE) procedure. Then, on the basis of the results, a decision is made with respect to counterfill. The entire cycle is then repeated until all sections are stable. Finally, once a satisfactory LE design has been achieved, it is verified using Finite Element Limit Analysis (FELA), again with each sections being subjected to a stability analysis.

The automated system requires a computational time of a few hours for a complete pipeline route and thus offers very significant savings and provides the opportunity to modify and optimize the pipeline routing iteratively.

2 LIMIT EQUILIBRIUM ANALYSIS

The Limit Equilibrium Method (LEM) is widely used for slope stability analysis (see e.g. Duncan et al. 2006). The basic idea is as follows. First, postulate a failure mechanism given by a slip line. This is typically circular although it in principle is possible to use any shape. The assumed slip line defines the failing mass which is subsequently divided into vertical slices. At the interface defined by the slip line, a normal and shear stress, related to each other by the failure criterion, act. Next, different methods make different assumptions about the inter-slice forces. The two most common assumptions are those of the Ordinary Method of Slices and Bishop’s Method. In the former, all inter-slice forces are assumed zero. In the latter, only the inter-slice shear forces are assumed zero.

For the present application, the assumed failure line is as shown in Figure 5. It comprises two straight lines through the rock fill connected by an arc through the seabed. It involves three degrees of freedom as indicated in the figure and is in this way computationally equivalent to a standard circular slip line.

Experience shows that Bishop’s method is more accurate for relatively shallow failures while a weighted average of Ordinary and Bishop gives good estimates for deep seated failures. Hence, a weighted method which interpolates between the two former methods, depending on the depth of the failure mechanism, is employed.
3 FINITE ELEMENT LIMIT ANALYSIS

Finite element limit analysis (FELA) is well described in the literature (see e.g. Lyamin & Sloan 2002, Makrodimopoulos & Martin 2006, Krabbenhoft et al. 2007, Sloan 2013) and has by now reached a level of maturity where both 2D and 3D problems are solved on a routine basis (see e.g. Olsen & Krabbenhoft 2021). While classic limit analysis is concerned with the maximum magnitude of a given load that a structure can sustain, the present work is concerned with the complementary problem of determining the Factor of Safety (FOS), i.e. the factor by which the material strengths should be reduced in order to bring the system to a state of incipient collapse. This task may be carried out using repeated feasibility analysis (which is reminiscent of conventional limit analysis) as discussed by Krabbenhoft & Lyamin (2015). For the automated design system described, all design iterations are first carried out using LEM after which they are verified using FELA.

4 LEM VS FELA

In this section, the accuracy of the three different LE methods employed (Ordinary, Bishop and weighted Ordinary/Bishop) is briefly discussed.

As described in Section 2, the Ordinary Method of Slices tends to underestimate the FOS while Bishop’s method tends to provide overestimates. The idea behind the weighted Ordinary/Bishop method is to arrive an optimal compromise between the two.

To verify these assertions, results from a four different berms, comprising in total 96 sections have been analysed. The failure mechanisms, examples of which are shown in Figure 6, vary widely between the different sections. The FELA runs were conducted with a large number of elements such that the FOS obtained are practically exact. The correlation between these and the various LE FOS (in the following denoted $FOS_{LE}$) are as follows (see also Figure 6):

- Ordinary: $FOS = 1.09 \times FOS_{LE}$
- Weighted: $FOS = 0.97 \times FOS_{LE}$
- Bishop: $FOS = 0.90 \times FOS_{LE}$

Similarly, using a general purpose limit equilibrium package (SLOPE/W), Olsen et al. (2023) have found $FOS = 0.85 \times FOS_{LE}$. On the basis of these results we see that Bishop on average is unconservative, Ordinary is conservative and the weighted scheme is slightly unconservative, though superior to the former two.

![Figure 6. Examples of failure modes: LEM-Weighted (left) and FELA (right).](image-url)
5 EXAMPLE

In the following, we consider the berm shown in Figure 8. The berm is 50 m long and has a volume of 4,152 m$^3$. The seabed is a very soft clay with an undrained shear strength that increases linearly by 1.64 kPa/m from 1.26 kPa at the top surface. All results are obtained with a plugin that has been developed for the general purpose geotechnical FE program OPTUM GX.

LE-weighted stability analyses for sections along the berm gives the results shown in Figure 9 (negative values here indicate failure on the left while positive values indicate failure on the right).

We see that all FOS are less than 1. The critical (smallest) FOS occurs at $s = 18$ m. At this location, the failure is as shown in Figure 10.

This failure mode suggests the necessity of counterfill on the right on the berm. Adding such counterfill, and adjusting its length and height iteratively, eventually leads to the situation shown in Figure 11.

Rerunning all sections now provides the results shown in Figure 12.
The critical failure now occurs on the left at $s = -14$ m as shown in Figure 13. Note that the FOS is negative indicating failure on the left of the main berm.

Again counterfill is added to bring about the situation shown in Figure 14.

Re-running all sections gives the results shown in Figure 15.

The critical section is now located at $s = -24$ m with the failure mode shown in Figure 16.

The bearing type failure that occurs here can be countered with additional counterfill. However, while the failure shown in Figure 16 in this way is countered, a new one with $\text{FOS} < 1$ occurs in the upper part of the berm as shown in Figure 17.

This situation is again countered by adding counterfill, now in a way such that the part of the berm at which the failure occurs effectively is strengthened. This will lead to failure in another section which again is countered, and so on until all sections are stable. At this stage, after 7 iterations, the situation at $s = -24$ m (which is the critical section) is as shown in Figure 18.

At this final stage, the total volume of rock is $7,287 \, \text{m}^3$, a 76% increase from the volume of the main berm. The final design is shown in Figure 20.
The total time for the automated design of the present berm, including FELA verification is about 10 min.

6 CONCLUSIONS

A system for automated design of offshore rock berms has been described. An obvious extension is to include a 3D FELA verification, i.e. to operate on the actual geometry rather than selected 2D sections. This work is ongoing.

7 REFERENCES


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