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ROCK BERM DESIGN FOR PIPELINE STABILITY

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ABSTRACT

Stabilizing large diameter natural gas pipelines on the seabed against extreme hydrodynamic loading conditions has proven to be challenging in the northwest of Australia. Tropical storms, which affect the area annually between November and April, can generate wave heights exceeding 30 m and storm steady state currents of 2 m/s or more. Consequently, in shallow water depths, typically less than 40 – 60 m, subsea pipelines can be subjected to very high hydrodynamic loads, potentially causing significant lateral movement. To mitigate the risk of the pipeline suffering mechanical damage due to excessive lateral movement, quarried and graded rock is often dumped over the pipeline as a secondary stabilization solution.

In order to satisfy functional requirements, the rock berm must comprise of a sufficiently large rock grading size and berm volume to withstand the design hydrodynamic loading such that the pipeline cannot break out of the berm. The design of rock berms for pipeline secondary stabilization has traditionally followed a deterministic approach that uses empirical equations for preliminary rock sizing, followed by small-scale physical modeling for design verification and optimization. Whilst the traditional approach can be effective in producing a robust rock berm design, opportunities for further optimization are inhibited by a lack of available data and an imperfect understanding of the failure mechanisms.

This paper presents an overview of an improved approach for rock berm design optimization. A general overview of rock berms, the design principles, benefits and risks are also presented.

INTRODUCTION

Pipelines are typically the most effective method for transporting hydrocarbons from subsea wells to shore for processing. Extreme storm conditions and interference from shipping have resulted in numerous pipeline failures over the past few decades and must be carefully designed for.

It is common practice in the offshore industry to apply a concrete weight coating (CWC) to the pipeline to increase its submerged weight for on-bottom stability. The concrete coating, which is typically a few inches thick, also provides some degree of mechanical protection to the pipeline. However, there is a practical limit to how much weight coating can be applied to a pipeline, due to either the tension or handling capacity of the pipeline installation vessel or the handling capacity at the coating plant. In cases where the maximum coating thickness cannot provide the pipeline with a sufficient level of safety, a secondary stabilization method may have to be adopted.

One method for achieving pipeline secondary stabilization and/or accidental external impact protection is by dumping quarried rock over the pipeline. Depending on the water depth and armour rock grading size, a Side Dump Vessel (SDV) or Fall Pipe Vessel (FPV) is typically used to install a rubble mound near-bed structure that is commonly referred to as a rock berm.

This paper presents an overview of the rock berm design concept, the analytical design methods, comparison of analytical method with physical model test results, and recommendations for design practice

DESIGN PRINCIPLES

Pipeline Limit States

The purpose of the rock berm design is to provide an acceptable safety margin against relevant failure modes that may lead to exceedence of pipeline limit states.

Where the pipeline external concrete weight coating alone is not considered to provide an acceptable level of safety against a given failure mode, rock berms may be used as a secondary form of stabilisation. The relevant pipeline failure modes are loss of pipeline on-bottom stability and damage caused by external interference.

Excessive displacement caused by loss of on-bottom stability is considered a serviceability limit state (SLS) [2]. For pipeline stability, the rock berm must provide sufficient embedment such that the hydrodynamic loads acting on the pipeline do not exceed the resistance provided by the weight of the pipeline and pipe-rock frictional forces [2]. Otherwise, the pipeline may break-out of the berm and potentially resulting in excessive lateral displacement of the pipeline.

Rock Berm Design Requirements

In addition to the integrity of the pipeline, the designer must also consider the stability of the rock berm structure. The berm dimensions should be optimized to minimize the total life-cycle cost without sacrificing ability to satisfy the design requirements. The total life-cycle cost comprises rock supply and installation costs (CAPEX) and maintenance works (OPEX).

CAPEX is a function of armour rock grading size, the number of layers and the required rock quantity. A smaller rock grading can be much cheaper to produce and install, but is more susceptible to instability under hydrodynamic loading. Similarly, a rock berm with very steep side slopes will require a smaller rock quantity, but will be more prone to excessive deformation that may require maintenance works (OPEX).

Maintenance works may be required if the berm crest level drops below the minimum level required to satisfy the design requirements.. The decision on whether or not to perform maintenance works is made by the operator. Factors likely to be considered in the decision making process include the length of pipeline at risk, the frequency of vessel movement through that particular area and the remaining design life.

Damage Acceptance Criteria

An important aspect of rock berm design is specifying the acceptance criteria for the degree of berm damage allowed during the design hydrodynamic conditions.

The CIRIA Rock manual [1] provides guidance on calculating damage levels for this type of structure, but on the topic of acceptance criteria it states "*there is no strict guidance yet on which damage level should be applied in different situations*". This paper aims to provide guidance on this subject, which is based on a review of the relevant literature and experience designing secondary stabilization systems for several major gas pipeline projects over the last decade.

In the absence of strict guidance, the designer may choose to design a rock berm that is considered to be either statically stable or dynamically stable under the characteristic environmental load. Each approach has its own benefits and drawbacks.

Static Stability Design Approach

Statically stable structures are those where no or minor damage is allowed to under design conditions. The term *static* implies minimal movement of individual stones. Consequently, a berm cannot be considered to be statically stable where frequent or widespread rolling of stones occurs, even if such instability does not reduce the height of the berm

The armour rock grading is sized such that the individual rock particles are virtually stable during the characteristic environmental loading condition, which typically has a return period (RP) of 100 years. By ensuring that the risk of significant berm degradation occurring over the pipeline design life is sufficiently low, the pipeline stability requirements may be satisfied by a no-cover rock berm profile.

A no-cover rock berm profile can be defined as where the nominal height to which rock is placed adjacent to the pipeline is level with the crown of the pipeline. Figure 1 provides an example of a theoretical design profile for a no-cover rock berm.

A no-cover rock berm may have a defined crest width. Increasing the width of the berm provides improved stability against hydrodynamic loading at the expense of increased rock quantities.

The required side slope for stability is a function of the armour grading and hydrodynamic loading, however, side slopes of 1v:3h have been used successfully on past projects.

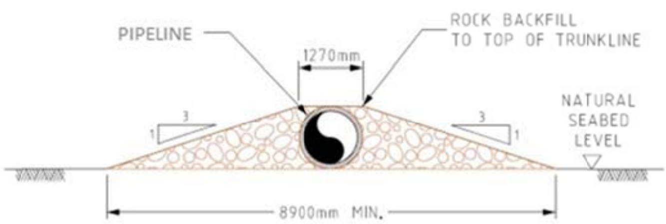

FIGURE 1 TYPICAL NO-COVER ROCK BERM DESIGN

A no-cover rock berm profile comprising a statically stable armour rock grading can produce a robust and cost-effective secondary stabilization solution. This approach is most suitable for water depths of less than 10 -15 meters, in cases where a reduced rock grading size does not offer significant benefits to the project. A statically stable design is also robust in that the level of berm degradation is not sensitive to the frequency and duration of storm events over the design life.

It should be noted that the theoretical rock berm profile shown in Figure 1 represents a minimum design profile that must be satisfied over the applicable length of the pipeline route. Due to inaccuracies of offshore rock placement, an additional quantity of rock must be allowed for overdumping to ensure that the minimum design profile is consistently achieved.

The accuracy of the rock placement is dependent on several factors such as the installation vessel, the water depth and the seastate conditions. Where berm installation is performed by a side dump vessel (SDV) in water depths greater than 20 m, the quantity of additional rock allowed for overdumping can often exceed the theoretical quantity based on the design berm profile. Consequently, the accuracy of rock placement has a significant impact on the cost of rock supply and installation and should be considered in the design selection process.

Dynamic Stability Design Approach

Dynamically stable rock berms are designed for significant movement of individual rock particles during the characteristic loading condition. This results in a development of the berm profile, with individual rock particles displaced by wave action until the transport capacity along the profile is reduced to a level such that an almost static profile is reached.

A key feature of a dynamically stable rock berm design is an allowable crest level drop that is built-in to the minimum design dimensions (Figure 2). The design may also feature a wider crest level to allow greater rolling of stones without crest level drop. By incorporating an additional rock quantity in the design to allow for some damage, the armour rock grading size may be reduced.

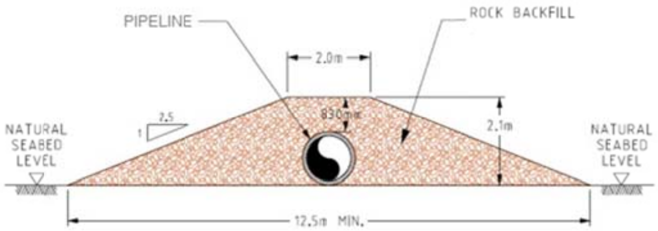

FIGURE 2 ROCK BERM DESIGN FOR DYNAMIC STABILITY

A potentially significant benefit of reducing the rock grading size is to allow berm installation with a Fall Pipe Vessel (FPV), which generally are not suitable for rock sizes larger than 250 – 300 mm. As FPV's typically have a much greater loading capacity and dumping accuracy than SDV's, reducing the rock grading to within the limits of a FPV can significant reduce the cost of installation when overdumping is factored in.

An additional benefit of reducing the armour rock grading size is the reduced risk of pipeline damage by falling armour rock particles during berm installation. In cases where this risk is considered unacceptable, a protective coating may be required to absorb impact energy.

The main drawback of this approach is the considerable level of uncertainty in predicting the degree of berm damage for a given hydrodynamic loading condition. This will be discussed in the following section of the paper. For this reason, physical model testing is typically performed to verify and potentially optimize a preliminary rock berm design.

DESIGN METHODS

Near-bed structures are submerged structures with a relatively low crest height compared to the water depth. The depth of submergence of these structures is sufficient to assume that wave breaking does not significantly affect the hydrodynamics around the structure. It is not uncommon for quarried rock to be dumped over a pipeline in very shallow water near the shore crossing, although the pipeline is typically laid in a trench such that the berm crest is not above the natural seabed level.

Compared to the stability of rock slopes of breakwaters and submerged structures, relatively scarce information is available on near-bed structures where currents and non-breaking waves form the primary design load. This section of the paper provides a brief literature review of design methods for both statically stable and dynamically stable rock berms under waves and currents.

Critical Shear Method

The traditional design method for hydraulic stability of rockfill is based on the incipient motion or critical shear concept proposed by Shields [5].

The stability of the rock berm is assessed by calculating the value of the Shields parameter, Ψ, which is a nondimensionalization of the shear stress and is given by Eq. 1

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$$\Psi = \frac{\tau}{(\rho_r - \rho_w) g D} \quad (1)$$

Where,

τ = bed shear stress

ρ_r = rock particle density

ρ_w = density of seawater

g = gravitational constant

D = rock particle diameter

The incipient motion method is based on the premise that ovement of individual rock particles is initiated when the Shields parameter Ψ, exceeds a critical value, Ψ_{cr}.

A pipeline designer can use this method to calculate the required stable rock particle size D for a given set of hydrodynamic loading conditions and rock density. To account for the fact that rock gradings produced in the quarry are not uniform, the characteristic rock particle size D in Shields's formula is commonly taken as the median nominal diameter D_{n50}. This infers that the largest 50% of rock particles by mass will remain stable for design loading condition, whilst the smaller rock particles may experience some limited movement.

This approach is an attractive method to use for preliminary rock sizing in design practice, because it provides a straightforward answer to the question of what rock grading size is required to ensure a stable rock berm design. However, the answer to this question is not so simple. The designer must decide on what value to assume for the critical Shields number, and how best to calculate the bed shear stress for a combination of waves and currents.

A key assumption of the Shields approach assumes that there is a clear threshold of motion that can be expressed as a critical shear stress. In reality the boundary is not clearly defined due to the stochastic nature of bed shear stress, protrusion, interlocking and rock particle size. This poses a dilemma for designers because the selection of critical Shields parameter, bed shear stress equation and the design wave height will all have a significant effect on the calculated rock size for stability. To maximize confidence in the results, design engineers should select a combination that provides good correlation with scale model test results.

CIRIA/CUR [1] recommends $\Psi_{cr} = 0.03$ when the shear stress is averaged over a full-wave period, and $\Psi_{cr} = 0.056$ when the instantaneous maximum shear stress is used, in order to get good agreement with the results of a set of scale model test results performed by Rance and Warren [8]

Van den Bos [6] conducted a comprehensive review of a number of analytical design methods for near-bed structures in waves and currents, including a quantitative analysis of the most promising methods against a dataset of scaled model tests. This included the critical shear stress method for a number of different wave-current interaction models and wave height parameters.

Van den Bos [6] concluded from his analyses that the critical stability method is not the most suitable method, because the stability of a single stone could not be related to the stability of the structure as a whole. However, the critical stability approach can be used to provide a conservative estimate of the required rock size, provided that the 1%-exceedence wave height, $H_{1\%}$, and peak period, T_p, is used to calculate the velocity at the berm crest. Use of the significant wave height, H_s, can seriously underestimate the governing shear stresses. A critical Shields parameter of $\Psi_{cr} = 0.030$ should be used in combination with the full wave period averaged shear stress, as per the recommendation by CIRIA/CUR [1].

Damaged Based Methods

An alternative approach to the critical shear method is to allow for a certain level of berm damage during extreme storm events, beyond which the pipeline may become unstable. Several equations have been proposed to calculate the damage to the structure as a function of the peak velocity (u), the weight of the stones and the number of waves (N). Most equations predict the damage in terms of the dimensionless erosion area S, which is expressed as the ratio between the eroded area A_e and the nominal stone diameter (D_{n50}).

$$S = \frac{A_e}{D_{n50}} \tag{2}$$

Where,

A_e is the erosion area of a cross-sectional berm profile

D_{n50} is the nominal rock particle size

Van Gent and Wallast [7] performed scale model tests of the stability of near-bed structures under a combination of waves and currents. Analysis of the test results lead to the following formula:

$$\frac{S}{\sqrt{N}} = 0.2\theta^3 \tag{3}$$

Where,

θ is a dimensionless mobility parameter, which is similar to the Shields parameter Ψ but a function of velocity at the berm crest level rather than shear stress, and is given by:

$$\theta = \frac{{u_0}^2}{g\Delta D_{n50}} \tag{4}$$

Where,

u_o is the maximum wave-induced orbital velocity at the berm crest level associated with the significant wave.

An important aspect of the equation is the accumulation of damage with the number of waves N, as this governs the sensitivity of the design to storm duration or multiple storms over its design life. Saers [4] conducted scaled model tests for irregular waves and suggests that the influence of time (i.e. the number of waves) could better be expressed with a logarithmic function instead of the square root function used by Van Gent and Wallast [7]. His design formula is:

$$\frac{S}{\log(N)} = 0.8\theta^{2.5} \tag{5}$$

Van den Bos [6] analyzed several datasets from scale model tests performed by Van Gent and Wallast [7], Saers [4] and others. He found that the correlation between the predicted damage and the observed damage could be improved on by making several modifications to the design equation, such as changing the relationship between S and N to $S \sim N^{0.3}$:

$$\frac{S^*}{N^{0.3}} = a \cdot (\theta_{hc1\%})^{1.6} \cdot m_0^{-0.6} \tag{6}$$

Where,

a is a model constant The most likely value for the model constant is a = 0.048, the upper bound is a = 0.12, the lower bound is a = 0.02.

$\theta_{hc1\%}$ is the dimensionless mobility parameter based on the peak velocity at the berm crest associated with $H_{1\%}$ and T_p.

m_0 is the berm side slope.

S* is the dimensionless erosion area per unit of crest width, B_c, and is given by Eq. 7:

$$S^* = \frac{A_e}{B_c D_{n50}} \tag{7}$$

A common feature of the various damaged based methods is that they do not include the influence of steady currents. Van Gent and Wallast [7] states that the influence of the current can be neglected within the following range: $U/u_o < 2.2$, where U is the depth averaged current velocity (m/s), for the following range of the mobility parameter: $0.15 < \theta < 3.5$.

It has been proposed earlier in this paper that the acceptable level of damage for a dynamically stable rock berm be specified in terms of an allowable crest level drop Δz. Consequently, it is necessary to relate the crest level drop to the calculated damage number S or erosion area A_e. The crest level drop for a given erosion area will depend on the proportion of material eroded from the berm slopes relative to the berm crest, which is not a constant.

Van den Bos [6] described four different damage profiles and compared the predicted values for each method with the measured values in datasets where both S and Δz were measured. A conservative estimate of the crest level drop can be obtained by assuming that a slice with a constant thickness is removed from the crest of the structure (Figure 3).

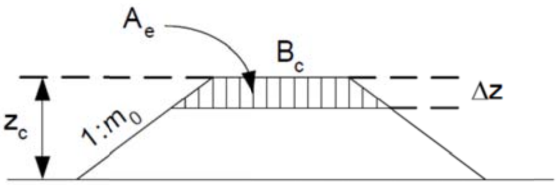

FIGURE 3: SLICED PROFILE [6]

Based on the 'sliced' profile, the crest level drop can be calculated as follows:

$$\Delta z = \frac{-B_c + \sqrt{B_c^2 + 4m_0 A_e}}{2m_0} \tag{8}$$

Physical model testing

Given the limitations of the various analytical design methods as well as the high consequences of failure, small-scale physical model testing is almost always performed to verify and optimize the rock berm design.

In addition assessing the response of the rock berm structure to the design hydrodynamic loads, physical modeling is a valuable tool for assessing the likelihood of lateral pipeline displacement. Given that the purpose of the rock berm is to provide pipeline stability, it is equally if not more important to understand the response of the pipeline.

When assessing the response of the design to the characteristic hydrodynamic loading condition, the allowable berm damage is usually limited such that it can be assumed the pipeline is not able to break-out of the berm. For more extreme conditions that are unlikely to occur during the design life of the pipeline, the performance of the rock berm may be judged on whether it can prevent pipeline lateral displacement

As is the case with the analytical methods for rock berm design, small-scale physical modeling is not without its own limitations and inaccuracies. This should be taken into account by the pipeline designer when specifying the design acceptance criteria. The designer should also aim to minimize the limitations of model testing by selecting an appropriate test facility, and by ensuring that all important design parameters have been scaled correctly.

There exists several types of test facilities that may be used for physical modeling of near-bed structures under waves and currents. Each type of facility has unique benefits and limitations, which are best suited to different environmental conditions.

A wave and current flume has traditionally been the most commonly used facility for this work [10]. It allows for quasi-2D modeling of structures exposed to long-crested waves and a co-linear steady current. The orientation of the model rock berm is typically perpendicular to the direction of waves and currents. Some key advantages of using this type of facility are availability and ease of model set-up and testing. Limitations are scaling effects for all but the largest flumes as well as end effects due boundary conditions created by the walls of the test section.

A wave basin has been utilized on several past projects for 3D model testing of secondary stabilization designs [9]. Both complex local bathymetry and directional waves can be modeled to provide a more realistic representation of the prototype conditions. It is also possible to assess multiple designs in a single test run. Disadvantages of using this type of facility include scaling effects, increased model set-up time and difficult to model steady currents.

DESIGN CASE STUDY

The design outcomes from a recent project are presented in this section of the paper to provide an example of how pipeline designers may select a rock berm design for secondary stabilization.

The preliminary berm designs were selected based on the empirical methods that have been discussed earlier in the paper. Physical modeling was performed to verify the preliminary designs and select a final design.

The paper will also provide some discussion of the test results, which will be compared against the predictions from analytical design methods.

Design Conditions

To provide a summary of the extreme hydrodynamic loading conditions, the design values of the most relevant metocean parameters are listed in Table 1.

TABLE 1: SUMMARY OF METOCEAN DESIGN VALUES

Parameter	Units	Return Period (years)		
		100	1,000	10,000
Still Water level	m	14.2	15.0	15.9
Significant wave height	m	6.5	6.9	7.3
Peak wave period	s	12.6	13.5	13.7
Zero crossing period	s	9.4	10.1	10.3
Peak significant wave orbital velocity	m/s	1.52	1.59	1.62
Steady current velocity	m/s	0.86	0.96	1.03

Acceptance Criteria

The following acceptance criteria were used for the rock berm model tests:

• 100 year RP event – the crest level may not drop below the crown of the pipeline. For the no-cover rock berm design, any crest level drop is unacceptable.

• 1,000 year RP event – some crest level drop is acceptable. This may result in the entire crown of the pipeline being visible. Sufficient embedment and sheltering should be provided by the rock berm such that pipeline instability does not occur.

Preliminary Berm Designs

Two rock berm design options were selected based on the results of empirical design methods and previous project experience.

A no-cover rock berm design, comprising a 300 mm D_{50} rock grading, a berm crest height and width equal to the pipeline outer diameter, and side slopes of 1v:3h (Figure 1) was chosen as the base case design. The incipient motion approach was used to calculate the required rock grading for a statically structure under the 100 year return period tropical cyclonic conditions.

A full-cover rock berm profile, with a significantly smaller armour rock grading size that could be installed using a fall pipe vessel, was selected as an alternative design. The design comprised a 175 mm D_{50} rock grading, a berm height of 2.1 m, crest width of 2.0 m and side slopes of 1v:2.5h (Figure 2). Results from damage based methods indicated that design would be dynamically stable, with any damage to the berm likely to be within acceptable limits.

Physical Model Set-up

The two preliminary rock berm designs were modelled in a wave/current flume at a length scale of 1:35. The flume is 30 m long, 1.0 m wide and 1.5 m deep. At one end is a wave generator capable of generating regular and irregular waves. At the opposite end is a parabolic “beach” to absorb wave energy and create as little wave reflections as possible in the flume. Three wave gauges were positioned in the flume to measure the surface elevation at three points.

A 50 cm diameter return pipe, a flow impeller, and the various connections at either end created circulation flow in the flume/pipe system. With a flume water depth of 0.5 m, this system is capable of producing a current up to 60 cm/s. Currents were measured by an Acoustic Doppler Velocity meter, a prototype location 1.4 m above the sea bed and approximately 2 m in front of the toe of the structure.

Each berm design was tested for hydrodynamic loading conditions equivalent to a return period of 100 years, 1,000 years and 10,000 years. Each test was ran for 2,000 waves. In this case, 2,000 wave represents a storm duration of approximately 7 hours, which is typical of tests carried out for coastal structures.

Test Results

For both the base case and alternative rock berm designs, the observed level of berm degradation was within acceptable limits for all three loading conditions that were tested.

FIGURE 4: BERM PROFILE AFTER 100 YEAR RP TEST

FIGURE 5 : BERM PROFILE AFTER 1,000 YEAR RP TEST

As expected from the relevant theory, movement of individual stones occurred much more frequently for the 175 mm D_{50} rock grading than for the 300 mm D_{50} rock grading. Despite widespread rolling of stones back and forth over the structure, less than 5% of stones were transported away from the structure and crest level drop was within acceptable limits for all tests (Figures 4 and 5). The response of the model rock berm to the test conditions provided a good example of a dynamically stable structure.

Design Outcome

Based on the results, both designs were considered to have a similar level of reliability. The alternative design was recommended based on the assumption that it could be installed using a Fall Pipe Vessel (FPV) For this particular project, the closest suitable rock load-out facility was located approximately 150 nautical miles from site. Potential for significant CAPEX savings existed by reducing the armour rock grading size to within the limits of a large FPV, which has significant advantages over most SDV's in terms of loading capacity, sailing speed and dumping accuracy.

Comparison of Test Results with Empirical Methods

To investigate the effectiveness of the different analytical methods described in this paper, the predicted crest level drop has been calculated over a range of velocities.

Both the predicted and measured values (at prototype scale) are based on the alternative rock berm design with a 175 mm D_{50} rock grading, a rock density of 2600 kg/m^3 and a storm duration of 2000 waves.

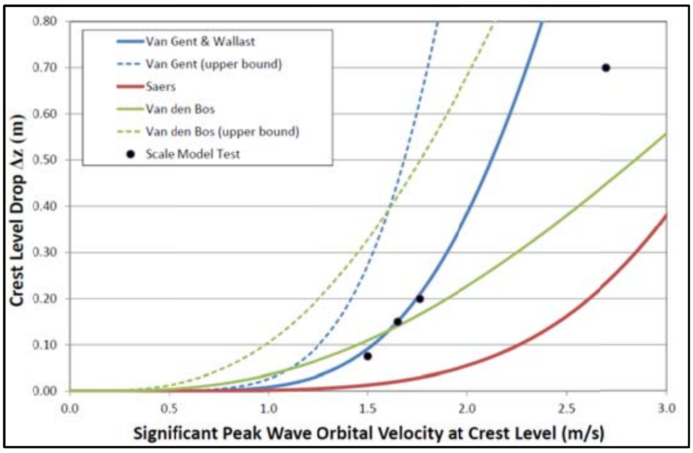

FIGURE 6: CREST LEVEL DROP OBSERVED DURING PHYSICAL MODEL TESTING AGAINST PREDICTED VALUES.

The academic value of this scale model test program is limited by the number of tests and the lack of instrumentation to accurately measure the change in berm profile. Consequently, the comparison between the predicted and observed crest level drop cannot provide any definitive conclusion on the most suitable damaged based method for design practice. However, Figure 6 does show a reasonably good correlation between the observed crest level drop and the most likely values predicted by the methods from both Van den Bos [6] and Van Gent and Wallast [7].

GUIDANCE FOR DESIGN PRACTICE

This paper has outlined several methods for modeling the response of a rock berm design to determine whether the structure can be expected to satisfy its design requirements.

In most cases it is relatively simple to ensure that a rock berm structure satisfies the project requirements during its lifetime in terms of acceptable failure rates and cost, given that the designer has a large enough budget to work with. It is much more difficult to demonstrate that the structure represents the economic optimal design.

Simple rules for optimizing the cost of rock berms - such as minimizing rock volume at the expense of a larger rock grading or vice-versa - are not universally applicable to every project. The economic optimal design is likely to change for each new project, even for cases where the environmental conditions are identical. This is due to variations in construction related factors such as the distance between site and rock load-out, or the cost and limitations of available installation vessels.

As a general rule, it is recommended that both statically stable and dynamically stable rock berm designs be considered in the design optimization process. The critical stability method can be used to determine the minimum armour rock grading size to ensure a statically stable rock berm with the minimum design height and width. A damage based method can be used to predict the required berm dimensions based on an armour rock grading size that is small enough to be reliably installed with a fall pipe vessel, and/or safely dumped on an unprotected pipeline. Both design methods should use the characteristic loading conditions with a return period of 100 years, as well as the minimum density of rock that can be expected from the quarry.

This process may lead to two or three different rock berm designs, of which one should be selected on the basis of reliability and cost. Physical model testing is often a worthwhile undertaking to gain an increased understanding of the reliability of each design under different environmental conditions including wave height and water depth. It is generally recommended to perform physical model testing in order to verify that the structure responds acceptably to the design loading conditions. Scale model testing is also useful for investigating the sensitivity of the design to more onerous conditions and to different angles of wave attack.

As part of the design selection process, the cost of rock supply, load-out and installation should be estimated for each rock berm design option. The total quantity of rock must be calculated by considering the minimum design volume, the necessary overdump allowance and the bulk density of the rock berm structure. Productivity rates are estimated based on expected values for the vessel loading capacity, rock dumping cycle time and expected standby time due to bad weather, crew change and breakdown.

The cycle time for rock dumping should by estimated by considering the average time taken for each step in the installation cycle, as shown in Figure 7.

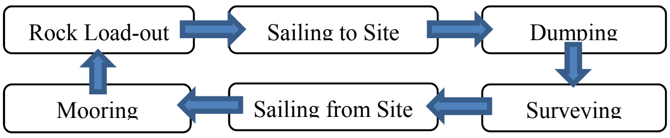

FIGURE 7 FLOW CHART OF ROCK DUMPING CYCLE

Information on vessel costs and productivity rates are based on past project experience and input from rock dumping contractors.

OPPORTUNITIES FOR IMPROVEMENT

Significant room for improvement remains for predicting the rock berm response to different environmental conditions, and for quantifying cost and risk to reach the optimal design.

Opportunities for improvement are currently limited by lack of quality data from scale model tests, and more so from berms in the field.

Damage based equations are still evolving with new research. As new sets of test data become available, the model constants are likely to change and new parameters may be introduced. It is important to note that the equations are largely based on scale model tests rather than prototype tests/observations. It is understood that scaling factors tend to have a conservative influence on the results, but this is difficult to quantify. Additional prototype data would provide increased confidence in using these equations for design optimization.

CONCLUSION

This paper has provided guidance on designing rock berms for pipeline stability. Damage acceptance criteria have been suggested and the potential benefits of designing for both static stability and dynamic stability have been discussed.

The most suitable design approach for a particular project will largely be governed by the relative influences of rock grading size and rock volume on the cost of rock supply and rock dumping. Consequently, the designer should identify step-changes in the cost of rock installation with increasing armour rock grading size.

Several empirical methods for preliminary rock sizing have been presented. It is important to consider the limitations of each empirical method, which are typically more applicable for either wave dominant or current dominant conditions. Given the limitations of empirical methods, it is generally recommended to perform physical model testing in order to qualitatively assess the design reliability.

REFERENCES

[1] CIRIA (2007), *The Rock Manual – The use of rock in hydraulic engineering (Second addition)*, CIRIA C683, London

[2] Det Norske Veritas, 2010a, "*On-Bottom Stability of Submarine Pipelines*", Recommended Practice DNV RP-F109

[3] Det Norske Veritas, 2010b, "*Risk Assessment of Pipeline Protection*", Recommended Practice DNV RP-F107

[4] Saers, W. (2005) *Erosion of rubble mound near-bed structures under irregular waves,* TU Delft, Delft, The Netherlands, MSc thesis

[5] Shields, A. (1936) *Anwendung der Aenhnlichkeitsmechanik und der Turbulenzforschung auf die Geschiebebewegung, Mitteilungen der Preuszichen Versuchsanhalt fuer Wasserbau und Schiffbau (no 26)*, Berlin, Germany, PhD thesis

[6] Van den Bos, J. P. (2006) *Design of granular near-bed structures in waves and currents*, MSC Thesis, Delft University of Technology.

[7] Van Gent, M. R. A., and Wallast, I (2001) *Stability of near-bed structures and bed protections; analysis of physical model tests with waves and currents*, WL|Delft Hydraulics, Delft, The Netherlands, Delft Cluster Report HC030204H3804

[8] Rance, P.J. and Warren, N. F. (1968) *The Threshold movement of coarse material in oscillatory flow* In: J W Johnson (ed), Proc 11th int conf in coastal engg, London.

[9] Jayewardene. I.F.W., Modra, B., Campbell., D.R., Chamizo., D. J. (2011), *2D and 3D Physical Modelling of Current and Wave Interactions for Rock Berm Design,* Proc. Coasts and Ports 2011.