Proceedings of ISFOG 2025
5TH INTERNATIONAL SYMPOSIUM ON
FRONTIERS IN OFFSHORE GEOTECHNICS
Nantes, France | June 9-13 2025

ISBN 978-2-85782-758-0

Finite Element Analysis of Subsea Expansive Anchor Piles in Rock Seabed

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ABSTRACT: An innovative anchor template aimed at rock seabed formations for the floating wind market is proposed. GRIP (product name) anchor comprises a subsea structural template with a skirted mud mat forming a pile cap accommodating three expansion anchor piles comprised of Circular Hollow Sections (CHS). In the proposed concept, a segment of the pile is expanded to form a bulb, creating a preload between the pile exterior surface and the drilled surface of the rock cavity. This expanding mechanism creates friction force as well as the mechanical interlock between the pile and rock, resisting axial loads. This article presents preliminary Finite Element Analysis (FEA) of pile expansion and axial pullout at the concept stage. A 2D axisymmetric model is created with S355 steel pile with conceptual dimensions and assumed rock properties. The FEA consists of two stages: 1) expansion of the pile inside the drilled rock hole during installation and 2) axial pullout during design life. A parametric study is conducted to investigate the effects of the pile diameter and the number of expansion points in the pile. This ascertains that the expanded bulb remains in the cavity formed in the rock after unloading. For the considered rock and pile properties, the axial capacity is approximately 7MN and 10MN for Ø406mm and Ø610mm CHS piles, respectively. There is a little change in the load capacity with the number of expansion points of the pile as the failure is governed by the yielding of the steel CHS pile. The increase in capacity is directly proportional to the increase in the diameter of the pile for the same wall thickness. FEA of the proposed expansive anchor piles in rock formations provides promising results with respect to axial capacity which needs to be verified with an experimental study.

Keywords: rock anchor; floating wind; expansion pile

1 INTRODUCTION

An innovative anchor template (namely GRIP anchor) aimed at rock seabed formations for the floating wind market is proposed in this paper. The anchor comprises of a subsea structural template with a skirted mud mat forming a pile cap accommodating three expansion anchor piles comprised of Circular Hollow Sections (CHS) as shown in Figure 1.

In the proposed concept of subsea expanding piles, a segment of the pile is expanded to form a bulb, creating an expanded interlock into the drilled surface of the rock cavity. This expanding mechanism creates friction force as well as the mechanical interlock between the pile and rock, resisting axial loads. The working principle is similar to expandable rock anchor bolts used in tunnels(Wen, et al.,2016), but under subsea conditions, a mechanical system will be necessary to expand the pile. A similar concept of expansive pile/anchor is adopted in recent studies for both cohesive and non-cohesive soil (Junghoon K. et al.,2022, Kuroyanagi, 2019).

The pile cap subsea structure comprises of a skirted mud-mat template which is extensively used for supporting subsea structures in the industry. The skirted mud-mat structure mobilises the soil plug trapped by the skirt against horizontal shear and enhances the shear capacity of the expansion piles. The subsea structure also prevents pile head rotation and acts as a load distribution structure between the piles.

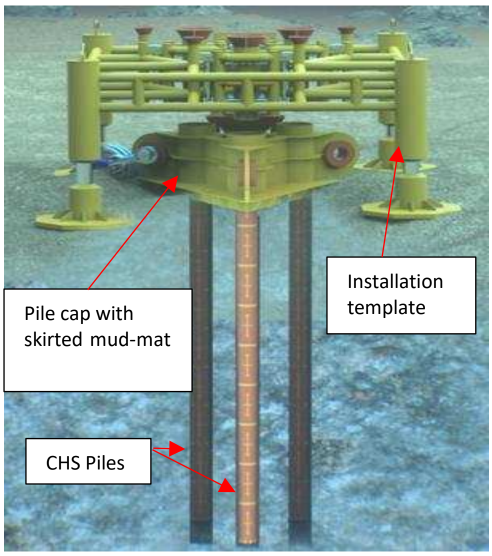

Figure 1: Anchor subsea structure and installation template

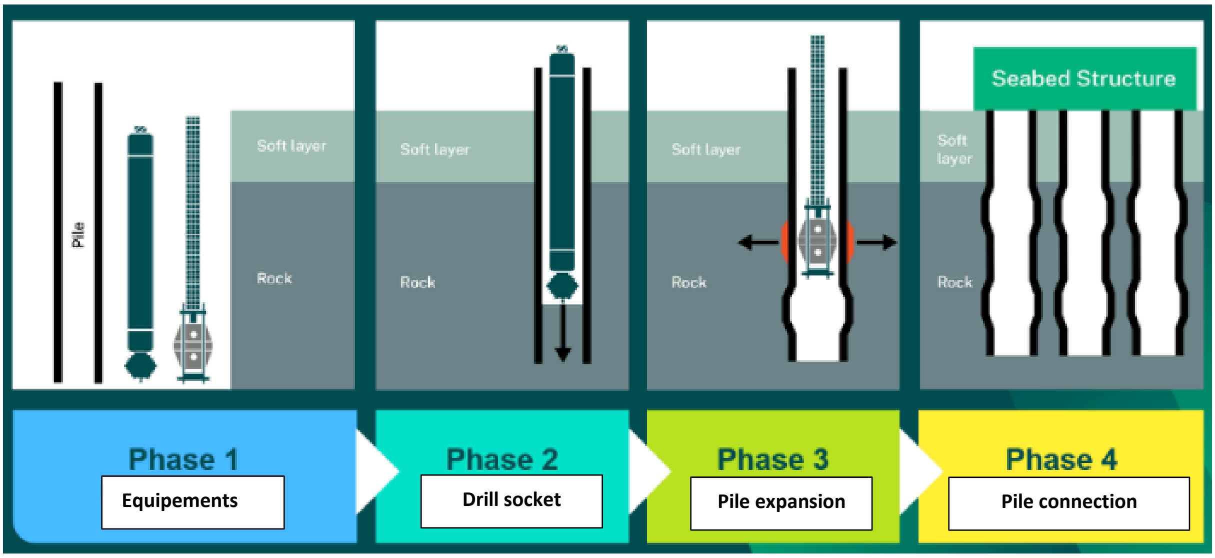

Figure 2:Grip anchor principle and installation sequence

Figure 2 presents a schematic installation method for the GRIP anchor. Ignoring the mobilisation step, which is part of the installation method, the first step (i.e. phase 2) is the drilling step wherein a CHS pile is placed inside the drilled cavity. The second step (i.e. phase 3) is performing pile expansion locally to engage the CHS into the drilled rock. This results in locking the CHS into the rock as well as the subsea structure without the use of grouting.

The present paper investigates the pile/rock interaction for a single pile.

2 FEA MODEL AND ASSUMPTIONS

A static FEA is carried out considering the geometric nonlinearity using the Abaqus Standard implicit solver (2023).

A schematic diagram of the axisymmetric model with dimensions for 406mm diameter and 12.5mm thick CHS is shown in Figure 3. The interactiob between the pile rock is considered in this analysis. The subsoil layer and skirted mud mat are conservatively ignored since they have a positive influence on the axial capacity. The clearance between the pile and the drilled rock surface is assumed to be 10mm.

At this preliminary stage, soil and rock properties are assumed. It is intended to refine this with an appropriate soil/rock design soil properties in the next stage reflecting data for a specific site. Table 1 reports assumed material parameters and nonlinear constitutive models. Steel S355 which is widely used for offshore structures is selected for pile. In modelling the rock properties, Concrete Damage Plasticity (CDP) model is used. A simplification is assumed by approximating the rock behaviour to Grade 50 concrete in the absence of damage parameters for rock. Details on deriving damage parameters of different concrete strengths can be found in literature (Milad, et al., 2017).

The analysis consists of 5 steps; initial step, 2 expansion stages (each at %50 of required total epansion), unloading and axial pullout. For each step, the initial time increment is 0.001s and the minimum time increment is 1E-12s. To further reduce the numerical convergence issues faced with the CDP model, automatic stabilisation is applied to the whole model.

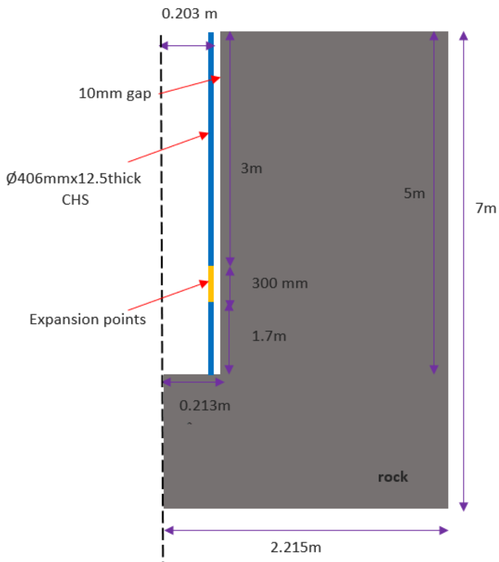

Figure 3. Schematic CHS pile/rock representation

Table 1: Material properties and constitutive behaviour

Material Parameters	Rock	Steel
Density	2323 kg/m^3	7850 kg/m^3

Constitutive behaviour	Concrete Damage Plasticity (CDP) model	Metal Elastoplastic
Modulus of Elasticity	34 GPa	205 GPa
Poisson ratio	0.25	0.3
Yield stress	25.7 MPa (compression)	355 MPa (tension)
Ultimate strength	51.3 MPa (compression)	501 MPa (tension)
Tensile strength	2.86 MPa (tension)	n/a
Dilation angle	38°	n/a
Friction coefficient	0.47	

Figure 4 displays the loads and boundary conditions of the axisymmetric model. The horizontal dimension of the rock is selected as almost ten times the radius of the pile to prevent boundary effects. The permanent boundary conditions are applied in the initial step.

The pile is expanded in two steps over 330mm circumferential to embed the rock surface to reduce the convergence issues caused by establishing the contact. During step 2, 10mm radial displacement is applied to initiate the contact and during step 3, 10 mm further displacement is applied to penetrate the pile into the rock.

To physically represent the pile expansion in the field, an analytical field is utilised to specify the distribution of applied displacement along 330 mm length as shown in Figure 4. This further reduced the numerical issues such as mesh distortion caused by sudden load jumps. In addition, the force distribution presented in Figure 4, is close to reality when a section is locally expanded. This expansion shape also reduces the bounce back effect due to the arching action of the bulb.

During step 4, the expansion is removed to measure the bounce back of the expanded area. During step 5, 60mm vertical displacement is applied to derive the axial force displacement curve of the pile. Horizontal displacement and rotation of the top of the pile are prevented during the axial pullout as shown in Figure 4. This is to replicate the field conditions where the top part of the pile does not move horizontally during pullout since it is stiffened with the subsea structure.

Surface-to-surface contact is defined between pile and rock. A tangential behaviour is simulated with a penalty-friction function with a friction coefficient of 0.47 (Ziogos, et al., 2023). The normal behaviour is defined as a hard contact.

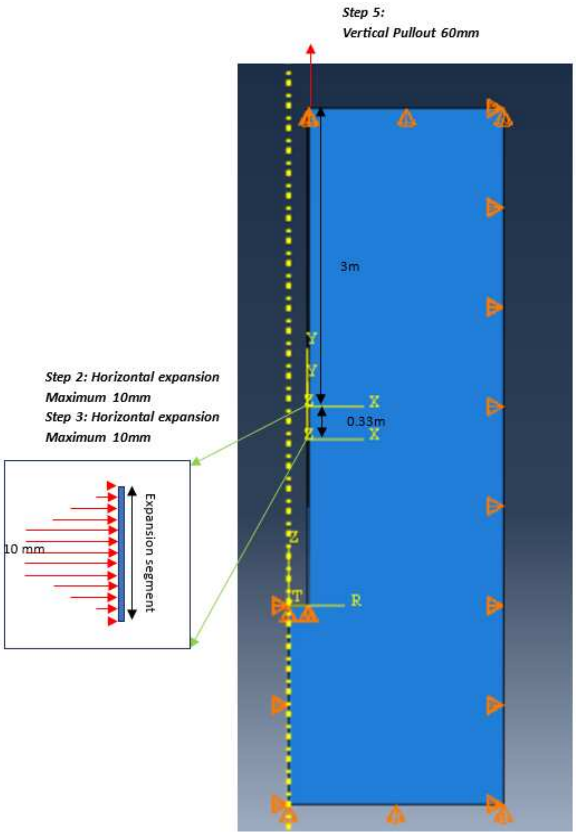

Figure 4: Boundary conditions of the FEA model

For both pile and rock, the element type used is 4-node bilinear axisymmetric quadrilateral, reduced integration and hourglass control (CAX4R). The element sizes are refined around the area of load application. The minimum element sizes are 3.125 mm in the pile and 25 mm in the rock.

To understand the influence of the pile diameter, the same analysis is repeated with 610 mm pile. Further, the effect of increasing the number of expansion points is studied by including three successive 330mm expansions 750mm apart from one another along the length of Ø406mm pile.

3 RESULTS OF PRELOAD ANALYSIS OF THE PILE (EXPANSION OF THE SLEEVE)

The expansive displacement is selected to have sufficient plastic deformation but prevent plastic hinge failure. For the considered expansive displacement, less than 40% (maximum nodal) bounce back is observed after unloading. The expanded bulb remains in the cavity formed in the concrete/rock after unloading as shown in Figure 5. The contact pressure at the rock steel interface increases with expansion and decreases considerably (about 80%) when the expansion force is removed.

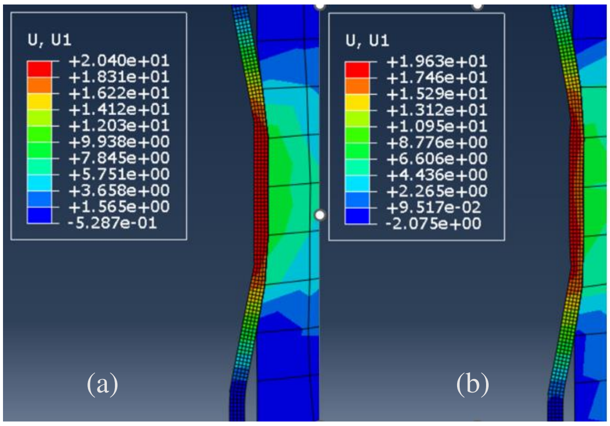

Figure 5: Horizontal displacement contours (mm) (a) after 20mm expansion (b) after unloading

Figure 6(a) displays the compressive damage parameter which is indicative of crushing failure. Figure 6(b) displays the tensile damage parameter which is indicative of cracking failure. Damage parameters are scalar values which indicates the amount of stiffness degradation and varies between 0 to 1. For the considered expansion and rock properties, damage propagation is not significant as it is local to the expansion location. The results are dependent on the rock constitutive model and parameters as well as the depth of the expansion point.

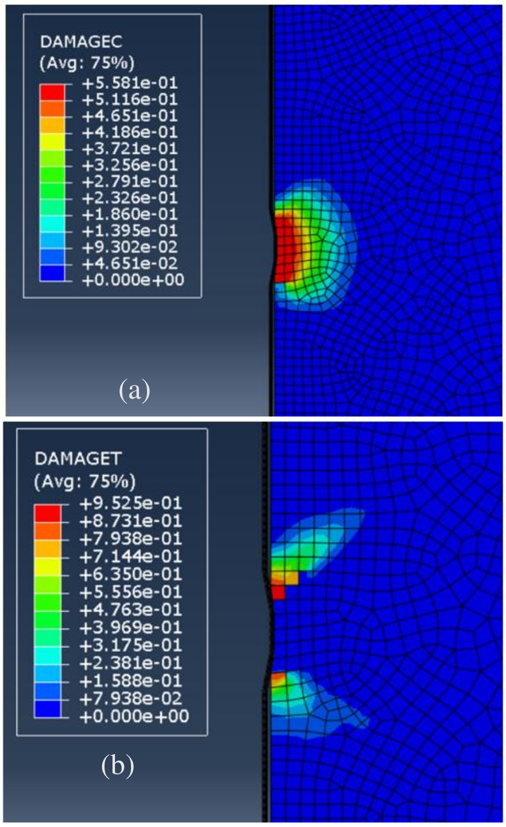

Figure 6: (a) Compression damage parameter (crushing) and (b) tensile damage parameter (cracking) after expansion.

4 RESULTS OF AXIAL PULLOUT OF PILE

For a single expansion bulb of 330mm, Ø406mm pile (CHS) indicates an axial capacity of approximately 6.8MN as shown in Figure 7. When the diameter is increased to 610mm, the capacity increases to 10MN. In both cases, the pile can be loaded up to the ductile failure of steel. The increase in capacity is directly proportional to the increase in diameter of the pile (i.e. 150%) for piles with the same wall thickness. However, it should be noted that the observed results depend on the assumed rock strength and the depth of the expansion point. Apparently, cone failure of the rock is prevented due to very high rock strength ultilised as well as the higher depth/diameter ratio of the expanded point.

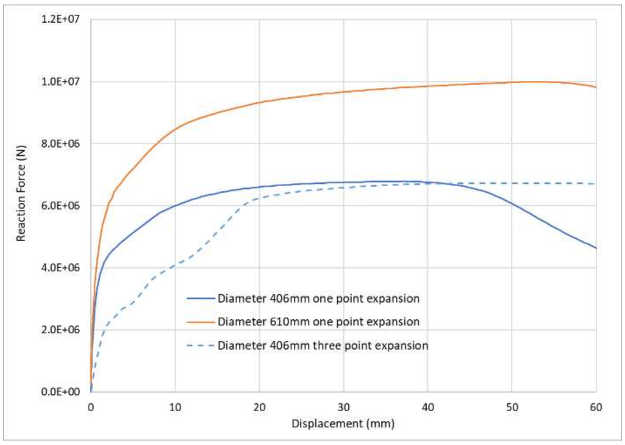

Figure 7: Vertical force vs displacement during pullout

The axial force is resisted both by friction and the mechanical interlock at the expanded bulb and the rock. i.e. The expanded diameter is larger than the drilled rock, thereby providing the mechanical interlock during the axial pullout.

To evalutate the effect of number of expansions, 3 expansions around 0.75m apart were induced in the model with Ø406mm pile. Figure 7 also displays the force displacement curve for the Ø406mm pile with three expansion points. As can be seen there is a little change in the load capacity, indicating failure load being govered by the yeilding of the steel pile.

5 SUMMARY AND CONCLUSIONS

The axial capacity of a novel anchor concept with an expansive sleeve is presented with FEA. Based on assumed rock and steel properties, the expanded bulb remains in the cavity formed in the concrete/rock after unloading with a maximum nodal bounce back of 40%.

For the considered rock and pile properties, the axial capacity is approximately 7MN and 10 MN for Ø406mm and Ø610mm CHS piles respectively. For the range of depth/diameter ratio and rock strengths considered in the current FEA, the possibility of rock conical failure can be eliminated.

The axial load capacity of the pile has not been significantly influenced by successive multiple expansion bulbs as the failure is governed by the yielding of the steel pile. However, the inclusion of more than one expansion point provides redundancy in terms of capacity.

6 LIMITATIONS AND FUTURE WORK

This study is a preliminary analysis of a concept design. Hence there are several limitations and room for improvements.

Assumed rock strength and damage properties are indicative and the results will be different for different rock properties.

The post-failure stress strain relation of CPD model introduces mesh sensitivity in the results. To circumvent this drawback, the fracture energy cracking criterion (for the tensile post-failure response) can be defined to capture the brittle fracture behaviour of rocks.

The rock is assumed to be intact whereas in field conditions joints, fissures and cracks can reduce the rock strength and the failure mechanism depends on joint orientations. An anisotropic rock constitutive model which accounts for rock mass properties will be explored in the next stage.

Most importantly, the results obtained from this FEA have to be verified with small scale laboratory experiments (1:8 scale) of pile sleeve expansion and axial pullout with different pseudo-rock strengths. Effects of loading rate and boundary conditions as well as different failure mechanisms can be observed during the scaled experiment. Furthermore, the material parameters of different rocks can be calibrated.

AUTHOR CONTRIBUTION STATEMENT

First Author: Finite Element Analysis, Writing the original draft. **Second Author**: Conceptual design, Methodology, Supervision.

ACKNOWLEDGEMENTS

The authors are grateful for the financial support provided by Venterra Group.

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