Rock Berm Restraint of an Untrenched Pipeline on Soft Clay

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ABSTRACT

This paper discusses soil structure interaction analyses performed to aid in the design of a rock berm to provide restraint to a surface laid pipeline in the case of full bore rupture of the line. The motivation behind this analysis related to the planned removal of an unexploded World War II mine, which was located against a high pressure gas transmission pipeline, in the UK Sector of the North Sea. Since soil conditions at the location consisted principally of soft clay, the analyses undertaken also highlighted the significant influence of the soft soil layer on rock berm stability under lateral pipeline loading, which is not well defined according to industry accepted design guidelines. As a result of the analysis presented this paper provides recommendations with regard to the design of a rock berm founded on soft clay.

Keywords: offshore pipeline, rock berm, pipeline restraint, soft clay.

1 INTRODUCTION

Rock dumping of untrenched, surface laid pipelines is generally undertaken to increase the lateral resistance of the pipeline to buckling. The construction of a rock berm may also be considered to increase the uplift resistance of the pipeline to withstand upheaval buckling. While it is important to understand the increased resistance to lateral loading provided by the rock berm (Kennedy et al. 2012) it is also equally important to understand the influence of the underlying soil, since where weaker soils are present this influence can be critical to the design sizing of the berm. The latter point is illustrated in this paper, through consideration of a case study example for an untrenched pipeline on soft clay.

An unexploded World War II mine (UXO) was discovered located against a high pressure gas transmission pipeline, in the UK Sector of the North Sea. It was decided that the risks associated with disturbing the mine were sufficiently low that it could be removed. However, a worst case scenario in which the mine is assumed to explode and sever the pipeline completely during removal was considered. Specifically, the response of one of the severed pipeline ends, under the action of the load due to the escaping gas, was analysed. The purpose of the analysis was to identify the suitability of application of a rock berm restraint to stabilise the pipeline in the event of an explosion and full bore rupture of the line.

Since soil conditions at the location consisted principally of soft clay, the analyses presented in this paper also highlight the significant influence of the soft soil layer on rock berm stability under lateral pipeline loading, which is not well defined according to industry accepted design guidelines. Finite Element analyses were performed to assess the uplift and lateral response of the pipeline, in plane strain. Using results of these analyses, three-dimensional (3D) pipeline soil interaction analyses were performed using proprietary software Sage Profile 3D (SP3D).

2 SOIL CONDITIONS

Soil conditions in the vicinity of the UXO location consist of about 1.5 m of very loose very silty sand overlying very soft clay to approximately 10 m depth. Stiffer soil is present beyond 10 m below seafloor (BSF).

The undrained shear strength versus depth for the clay layer is illustrated in Figure 1. The undrained shear strength was derived from CPT data using a cone factor, N_{kt} = 20 ($s_u = q_{net}/N_{kt}$). This factor was selected as it provided the best indicative fit with the available directly measured strength data, which included torvane and unconsolidated undrained (UU) triaxial test measurements on Vibrocore samples. The very soft clay layer can be characterised by an undrained shear strength which increases linearly with depth from s_u = 6 kPa at 1.5 m depth to s_u = 18 kPa at 10 m depth.

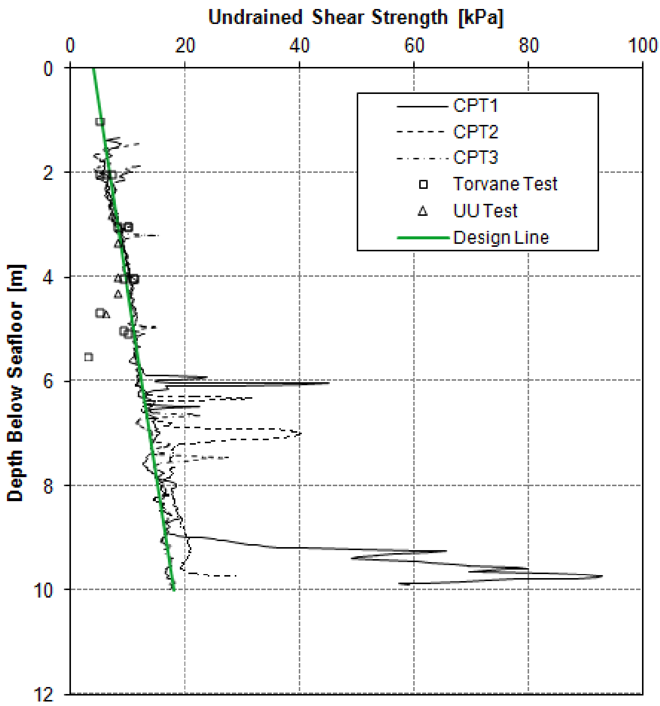


Figure 1. Undrained shear strength profile in vicinity of mine location

It was considered that, under the rapid loading conditions associated with a pipeline rupture, undrained conditions could prevail in the top sand layer, given the very high fines content of the sand. As an approximation, the undrained shear strength of the sand was obtained by extending the undrained shear strength profile defined for the clay to the mudline, as shown on Figure 1. In this case, the estimated undrained shear strength at mudline was approximately 4 kPa.

The selected soil stratigraphy and associated design soil parameters which were applied for the analyses described are summarised in Table 1.

Table 1. Design soil conditions

Depth [m]	Soil Type	Submerged Unit Weight [kN/m³]	Friction Angle [°]	Undrained shear strength [kPa]
0.0 to 1.5	Very loose silty SAND	8.5	20	4 (top) to 6 (bottom)
1.5 to 10.0	Very soft CLAY	8.0	-	6 (top) to 18 (bottom)

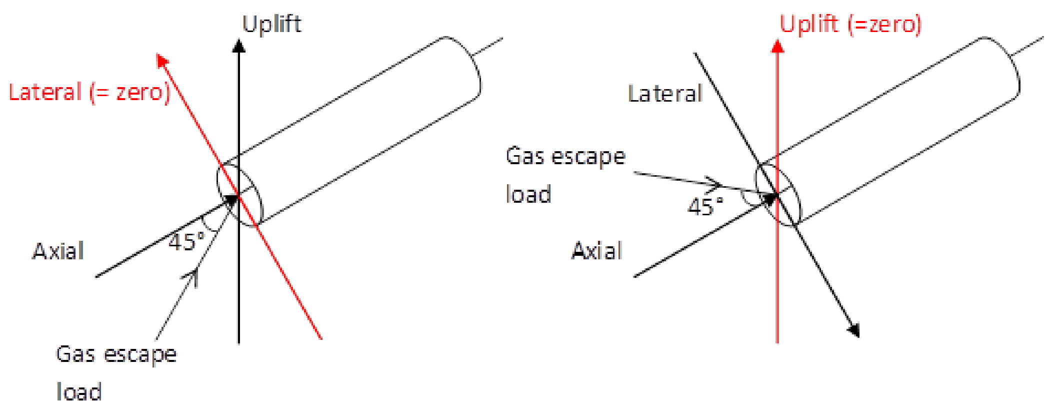
3 PIPELINE PROPERTIES & LOAD CASE

The pipeline is a high pressure gas transmission line, in the UK sector of the North Sea. It is a surface laid pipeline with minimum seabed embedment and has an outer diameter D = 0.91 m and a wall thickness t = 22 mm. The geometry and main properties of the pipeline are summarised in Table 2.

Table 2. Pipeline properties

Properties	Symbol	Unit	Value
Steel Grade	-	-	API 5L X60
Young's Modulus	E	(GPa)	207
Poisson's ratio	ν	-	0.3
Pipe Outer Diameter	D	(m)	0.9144
Pipe Inner Diameter	ID	(m)	0.87
Pipe Wall Thickness	t	(mm)	22

A compressive load of 1560 kN, related to gas escape at the rupture (UXO) location, was considered for the design of the rock berm. It was also assumed, nominally, that the compressive load would be inclined at 45° to the pipe axis, as illustrated in Figure 2. This corresponded to a compressive force of about 1120 kN acting along the pipe axis and a concurrent force of 1120 kN acting in the uplift or lateral direction. For the purposes of the rock berm design the uplift and lateral cases were considered independently.



(a) Uplift direction (b) Lateral direction

Figure 2. Loading directions for pipeline response

4 3D PIPELINE MODEL

4.1 Sage Profile 3D

The overall response of the pipeline under the action of the forces summarised in Section 3 was assessed using SP3D, in order to determine the size of the rock berm required to stabilise the pipeline. SP3D is an explicit Finite Element Analysis (FEA) software package for subsea pipeline analysis. Using a transient dynamic explicit solver, it can accurately simulate the response of the subsea pipe when subjected to hydrodynamic loading and operational conditions. A comprehensive overview of the use of SP3D for offshore pipeline design, installation and operation can be found in Van den Abeele et al. (2012).

The pipe/soil interaction was modelled using 'spring' elements in the vertical (downward), axial and lateral directions at intervals along the pipe. These elements incorporate simplified load transfer curves, which are often similar in format to the p-y or t-z models applied for analysis of pile behaviour. Plastic deformation of the pipeline was considered in the analysis and simulated using a Ramberg-Osgood stress-strain relationship.

A new feature in SP3D allows the user to perform an upheaval buckling (UHB) assessment. The feature provides a realistic simulation of UHB through incorporation of a user-defined backfill soil spring in addition to the three conventional pipe-soil interaction springs. The backfill spring captures the interaction between the pipeline upheaval movement and the mobilised backfill resistance through specification of a load transfer curve in the uplift direction. This feature was used to simulate the rock berm restraint in the uplift direction for the pipeline considered here.

4.2 Model Geometry

The analysis aimed to determine the rock berm size required to maintain the pipe in position should rupture occur. Various rock berm geometries were investigated. A berm on each side of the UXO location was required to maintain the pipe in position in case of explosion. Each berm was to be constructed at a distance of 25 m from the UXO. Since the problem geometry was symmetrical, only one berm was modelled in SP3D.

The model geometry started at the UXO location and considered a 1 km long pipe section. A straight pipe section laid on a perfectly flat seabed was assumed, as illustrated in Figure 3. The pipeline was modelled using 2 m long elements, corresponding to a total of around 500 nodes. Load transfer curves between the pipe, the seabed and the rock berm were attached at each node.

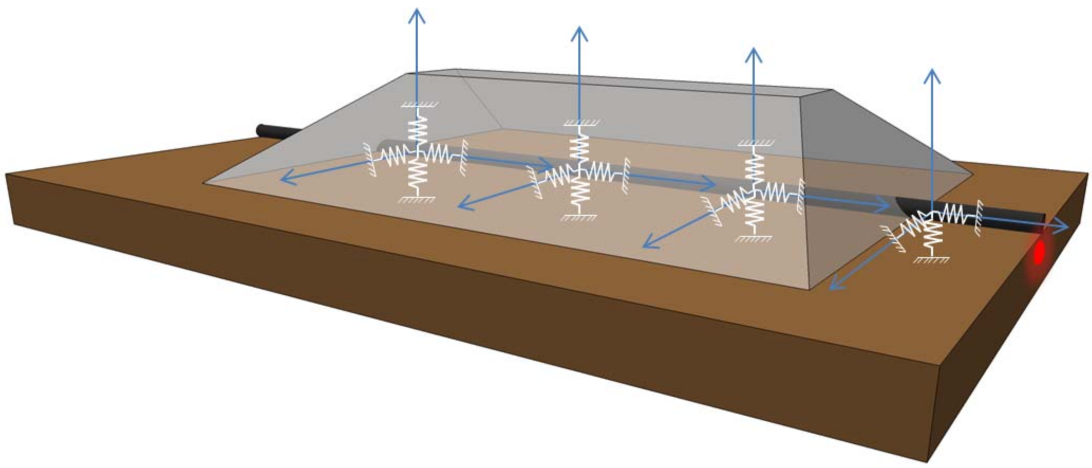


Figure 3. Model Illustration

4.3 Pipe/Soil Interaction

A simple bi-linear elasto-plastic response was considered in the axial direction. In the model a distinction was made between the uncovered and covered (by rock berm) sections to calculate the peak axial resistance. For the uncovered section of the pipeline, a friction factor of 0.5 and a mobilization displacement of 3 mm were assumed. For the covered section, the increased confinement of the pipeline was taken into account by applying the approach recommended by Schaminée et al (1990).

The vertical (downward) load transfer curves were derived according to the relationships presented by Aubeny et al. (2005), subsequently corrected to allow for the increased overburden pressure resulting from the weight of the rock berm in the covered sections of the pipeline.

Uplift and lateral load transfer curves were derived from the results of plane strain FEA using Plaxis 2D (Plaxis 2011), as described in Section 5.

5 2D PIPELINE RESPONSE

The load-displacement responses (and load transfer curves) of the buried pipeline in the uplift and lateral directions were derived using plane strain FEA. The 2D FEA model geometry, for a 5 m high rock berm with a side slope of 1V:4H, is shown in Figure 4. The overall lateral dimension of the model was 135 m and the depth of the clay layer was taken to 10 m below seafloor (BSF) assuming the soil profile presented in Table 1. These dimensions were checked to ensure that the model boundaries would have negligible effects on the results (i.e. the failure mechanisms developed under directional loading were fully encompassed by the boundaries of the model). The pipe was modelled using plate elements and corresponding to the dimensions presented in Table 2. Interfaces were specified along the pipe soil contact. Across these interfaces the limiting shear stress was defined as 50% of the value calculated in the adjacent rock fill or clay. For the uplift case a zero tension cut-off was specified along the lower surface of the pipe.

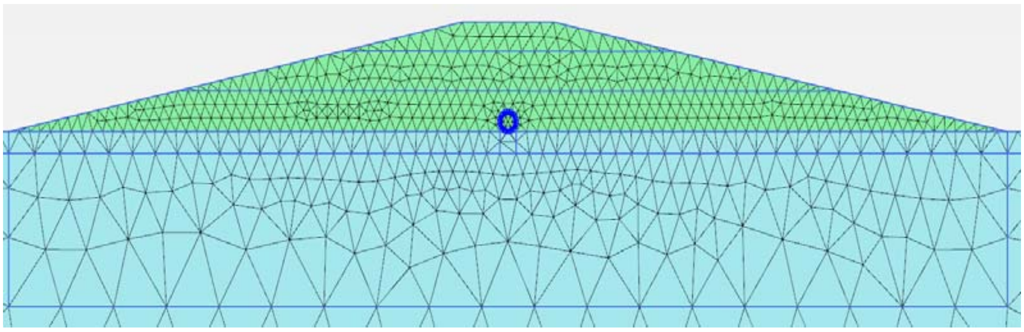


Figure 4. 2D FEA model geometry for a pipeline buried under a rock berm on soft clay

The soil and rock were modelled as an anisotropic elastic-perfectly-plastic continuum, with failure described by the Mohr-Coulomb yield criterion. The clay layer was assumed to behave undrained and was characterised by a cohesion equal to the undrained shear strength ($c'=s_u$, $\phi=0$) which varied with depth according to an initial undrained shear strength (s_{u0}) and an undrained strength gradient (s_{uz}). The elastic behaviour for the clay was defined by a Poisson's ratio ($\nu=0.49$) and by assuming a constant ratio of Young's modulus to undrained shear strength ($E/s_u=300$). The initial value of the coefficient of lateral earth pressure, K_0, was assumed to be 1.0 in the clay layer, which is considered appropriate for the characterisation of undrained strength applied (undrained shear strength derived by comparison to isotropic triaxial test data).

The rock fill layers were assumed to behave drained and the rock shear strength was characterised by an effective cohesion (c') and friction angle (ϕ). A conservative friction angle of 35° was selected and a nominal value of c' was specified to improve the calculation performance of the analysis. It has been shown that application of a fully associated flow rule ($\psi=\phi$) leads to unsafe results for the particular problem of pipeline uplift resistance (White et al, 2008). Therefore a non-associative flow rule was considered with an angle of dilation (ψ) of 15°. The elastic behaviour for the rock fill was defined by a Poisson's ratio (ν=0.25) and by a Young's modulus (E=90MPa). K_0 was assumed to be 0.5 within the rock berm.

5.1 Pipeline uplift resistance

Initially the unit uplift resistance, R, of the pipeline was calculated using the Trautmann-Pedersen vertical slip surface model (Pedersen and Jensen, 1988) in combination with an uplift factor f_p=0.5, which is the lowest recommended value for rock according to DNV-RP-F110. The recommendations of DNV-RP-F110 were then applied to obtain load transfer curves for input to SP3D.

In the vertical slip surface model, f_p is equal to K_0.tanϕ' by definition. Therefore, the equivalent uplift factor based on input parameters in the FEA assessment is only 0.35 (0.5.tan35°). However, White et al (2008) showed that the dilation angle influences the peak uplift resistance and proposed the following modified equation for the uplift factor:

$$f_p = \tan\psi + (\tan\phi - \tan\psi)\left[\frac{1+K_0}{2} - \frac{(1-K_0)\cos 2\psi}{2}\right] \qquad [1]$$

Values of 35° and 15° for the friction angle and dilation angle, respectively, as assumed in the FEA assessment, gives an equivalent uplift factor f_p=0.5 when using Eq. [1], which corresponds with the assumption made in application of the DNV vertical slip model.

The DNV curve for a rock fill depth of 4 m (i.e. 5 m rock berm height) is compared to the load displacement response obtained from the plane strain FEA in Figure 5. It is evident that the peak uplift resistance predicted by the DNV formulation (130 kN/m) is similar to that derived from the FEA (140 kN/m). However, the FEA response seems to be softer than the DNV curve. The uplift stiffness depends on the assumption made for the Young's modulus of the rock fill (E=90MPa was assumed for the case presented).

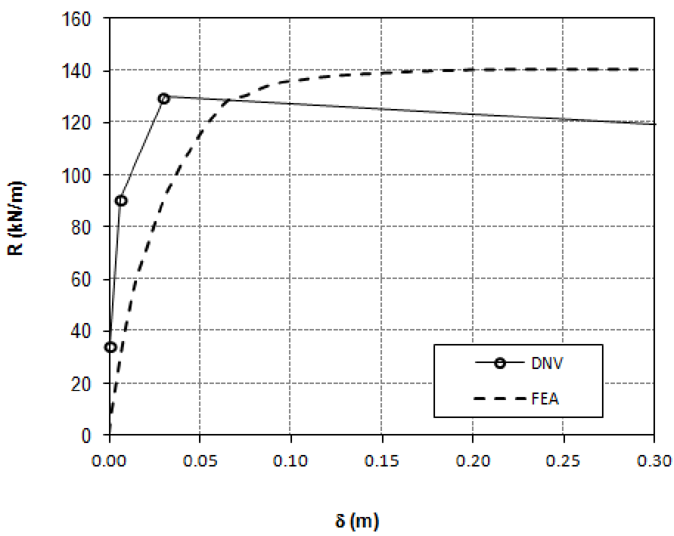


Figure 5. Comparison of pipe uplift load displacement, for a rock berm height of 5 m

5.2 Pipeline lateral resistance

2D FEA was also performed to check that the lateral resistance of the rock berm was at least equal to the uplift resistance. For each rock berm height considered a side slope of 1V:4H (approximately 14°) was applied. The side slope angle of the berm was selected to ensure an adequate factor of safety (1.3) based on the results of slope stability analysis (which presents an additional challenge to the design of a rock berm restraint on soft clay, but is not discussed further here). In addition, analysis presented by Kennedy et al. (2012), demonstrates that increasing the rock berm slope angle beyond 18° will significantly reduce the lateral resistance. Only the direction of the load acting on the pipe was changed between the uplift and lateral analyses.

The lateral failure mechanism obtained from the FEA for a 5 m high rock berm is illustrated in Figure 6. The associated ultimate lateral resistance for this case was approximately 50 kN/m, which is less than 40% of the uplift resistance for the same rock berm geometry. This value is also significantly less than the value of approximately 170 kN/m that might be inferred (ignoring the presence of the soft clay) based on the results of the parametric analyses presented by Kennedy et al. (2012), from which it was conservatively assumed that a lateral bearing capacity factor, N_q, of 4 would be representative for a rock friction angle of 35° and a side slope of 1V:4H.

It is evident then that the rock berm resistance to pipeline lateral movement is strongly affected by the presence of the soft clay layer. The lateral force introduced to the pipe leads to a bearing capacity failure of the berm in the soft clay to a depth of around 2 m BSF, as illustrated in Figure 6. It was concluded from the lateral analysis that the berm would need to be significantly widened to increase the lateral resistance to a value similar to the uplift resistance.

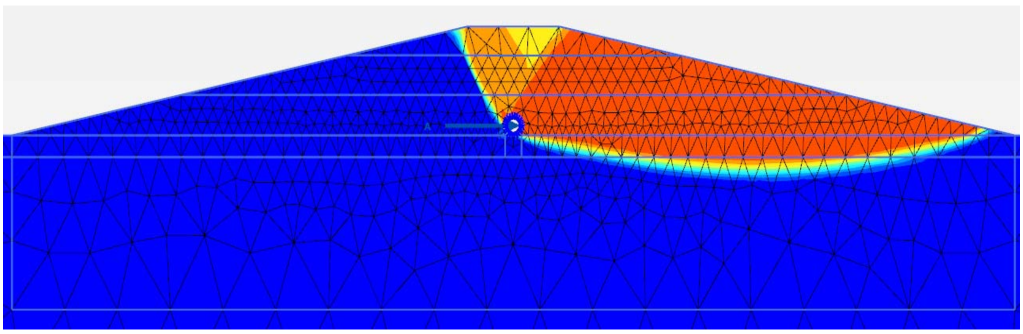


Figure 6. Lateral failure mechanism for a pipeline under a 5 m high rock berm with a side slope of 1V:4H

Considering the above conclusions, additional analyses were performed to assess how the lateral resistance of the rock berm increased with increasing base width. The results of these analyses are presented in Figure 7. It was concluded that a base width of about 90 m was required to reach a lateral resistance which is similar in magnitude to the uplift resistance for a 5 m high rock berm. This increased base width corresponds to a side slope of approximately 1V:9H (6°).

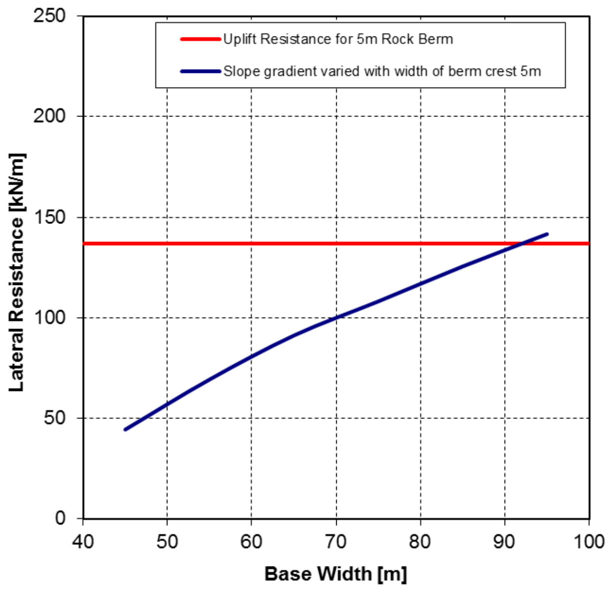


Figure 7. Lateral resistance of pipe under a 5 m high rock berm with increasing rock berm base width

6 3D PIPELINE RESPONSE

6.1 Analysis steps

Load transfer curves for the uplift and lateral load cases were extracted from the results of FEA, described in Section 5, and applied to the SP3D model.

The external load from gas escape, initially applied at the UXO location (25 m from the rock berm), was progressively increased up to pipeline failure for different rock berm configurations. Depending on the rock berm restraint considered, the observed failure could consist of the formation of a plastic hinge at either the toe end of the berm or under the crest of the berm, which could potentially trigger another gas escape within the berm. Therefore, the analyses were scheduled using the iterative process below:

- **Step 1:** First berm geometry, external load applied at the UXO location;
- **Step 2:** If Step 1 indicated pipe failure at the rock berm toe, run an additional analysis with the external load applied at the toe of the berm;
- **Step 3:** If Step 2 indicated pipe failure below the rock berm crest, run an additional analysis with the external load below the rock berm crest;
- **Step 4:** If Step 3 indicated pipe failure within the rock berm, increase the berm size and repeat from Step 1.

The iterative process was stopped when an optimised rock berm configuration leading to acceptable forces and displacements along the pipeline could be found.

6.2 Results

Figure 8 and Figure 9 present examples of results when the pipeline is subjected to a combined compressive and uplift force from gas escape applied at the rock berm toe (Figure 8) and below the rock berm crest (Figure 9) for 6 m high rock cover. In these figures, sets of three graphs are presented describing the pipe displacement, the mobilised rock fill resistance and the pipe bending moment. Note that the forces indicated on the graphs are the magnitude of the compressive (axial) and uplift components of the gas escape force. The resultant force can be obtained by multiplying these forces by 1/cos(45°), see Section 3.

For a 4 m rock cover a summary of the SP3D analysis iterations is provided in Table 3. The Step 1 analysis showed that the pipeline was failing at the rock berm toe due to excessive bending moment. In this case, the force that can be sustained was much less than the design force of 1120 kN. Increasing the rock berm height to 5 m does not help in this case because the failure occurs at the toe of the berm.

As already discussed in Section 6.1, a pipe failure at the berm toe could potentially lead to another gas escape at that location. When the gas escape force is applied at the toe of the berm, another pipeline failure is observed in the berm front slope because the uplift resistance is reduced compared to the available uplift resistance in the berm itself.

The next critical condition is when the gas escape force applied under the rock berm crest is able to unzip the pipeline by mobilising the full uplift resistance within the rock berm. The simulation showed that 4 m rock cover would be enough to maintain the pipe in position. However, the factor of safety is only marginally above one and a slight increase in gas escape force causes pipeline rupture. As also shown in Table 3, a 5 m rock cover (6 m high berm) provides a more satisfactory factor of safety and a bending moment limited to about 3000 kNm, which is approximately 50% of the moment required to initial plastic yield.

Table 3. Summary of 3D pipeline response analyses

Cover Height (m)	Location of Applied Force	Non-failure State		Failure Sate		Remarks
		Applied Force (kN)	Bending Moment (kNm)	Applied Force (kN)	Bending Moment (kNm)	
4	at 25 m from toe	200	5200	400	>8000	Rupture at berm toe
	at toe	600	5000	800	>8000	Rupture in berm slope
	below crest	1120	5000	1200	>8000	Design force is resisted with limited margin
5	at 25 m from toe	200	5200	400	>8000	Rupture at berm toe
	at toe	600	4500	800	>8000	Rupture in berm slope
	below crest	1200	4000	-	-	Design force is resisted

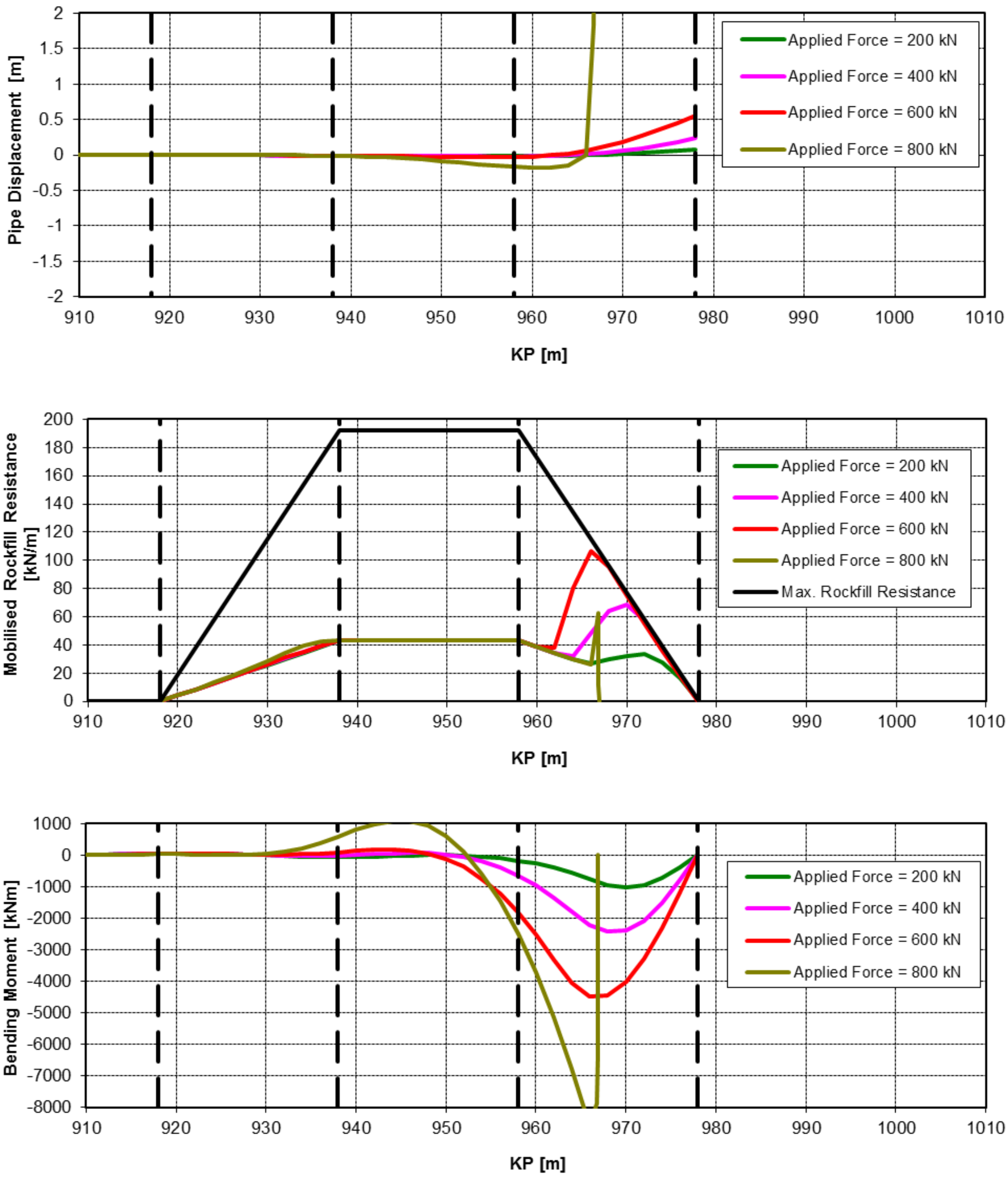


Figure 8. Example results for 6 m high rock berm, force applied at rock berm toe

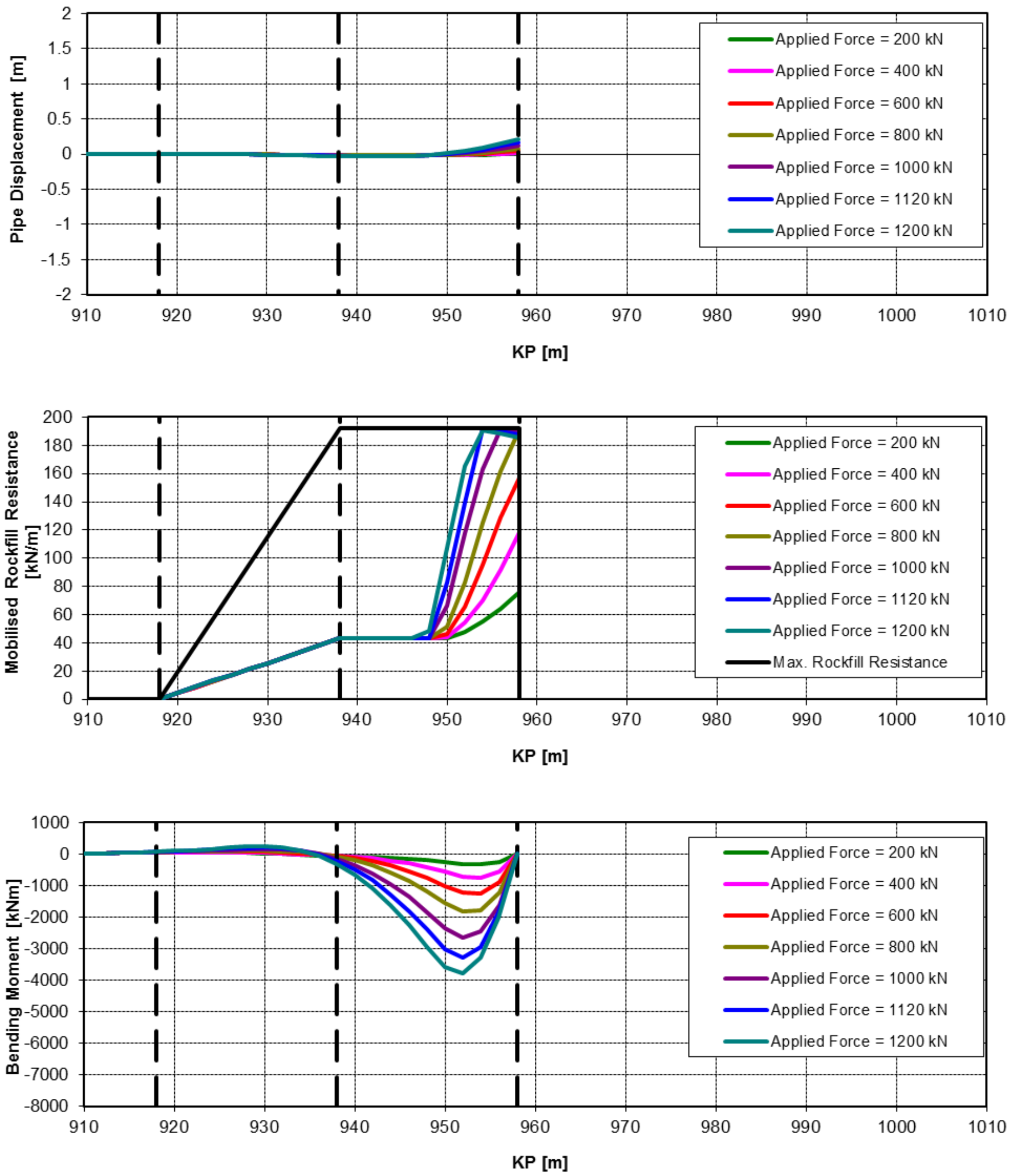


Figure 9. Example results for 6 m high rock berm, force applied below rock berm crest

7 SUMMARY AND CONCLUSIONS

This paper has presented the results of soil structure interaction analyses performed to aid in the design of a rock berm to provide restraint to a surface laid pipeline in the case of full bore rupture of the line. The influence of the soft shallow soils present at the location on the lateral resistance of the buried pipeline section has been highlighted as part of the analysis presented. It has been demonstrated that 3D pipeline response analysis provides a convenient tool to optimise the rock berm design, through application of an iterative analysis process which considers soil/pipe/berm interaction and the potential for failure points to develop at different pipeline locations, both external to and within the rock cover. The main conclusions of the analyses presented are summarised as follows:

- Upheaval buckling problems are conveniently analysed in Sage Profile 3D (SP3D) through inclusion of a backfill soil spring in addition to the three conventional pipe-soil interaction springs.
- Based on a no tension case assumption, the recommendations of DNV-RP-F110 and application of a vertical slip surface model (Pedersen and Jensen, 1988) provides uplift resistance estimates which show reasonable agreement with the ultimate uplift resistance derived from the results of 2D plane strain FEA. This conclusion is considered in light of the recommendations of White et al (2008), in relation to the influence of dilation angle on pipeline uplift resistance.
- The lateral resistance of rock-dumped pipelines is strongly affected by the presence of a soft soil profile below the rock berm. Lateral loading of the pipe leads to a bearing capacity failure in the soft soil which significantly reduces the ultimate lateral resistance. For the example case presented the lateral resistance may be only 30% of the predicted resistance where the soft soil layer is not considered.
- When considering the ability of a rock berm to provide restraint to a surface laid pipeline in the case of a full bore rupture, potential failure of the pipeline at the toe of the rock berm and below the crest of the rock berm (where the full backfill resistance is first mobilised) should be considered. Where inadequate restraint is present at these locations the pipeline may “unzip” itself from the berm through a series of progressive failures.

The findings of the study highlight that FEA is an important tool in optimising the design of a rock berm to increase the lateral resistance of a surface laid pipeline. Particular care should be taken in adequately modelling the underlying soil profile as well as the rock characteristics in any analysis performed.

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