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Drainage effect on the uplift resistance of buried offshore pipelines

Effet du drainage sur la résistance au soulèvement des pipelines offshore enfouis

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ABSTRACT: The design and construction of sub-sea pipelines accounts for a substantial portion of the capital expenditure of offshore infrastructure. The main design parameters required for the design of a pipeline against upheaval buckling are peak uplift resistance and corresponding mobilised displacement. These factors are influenced by the pipe diameter and burial depth as well as soil characteristics like frictional resistance and dilation. Recent experimental studies have revealed that the uplift resistance of pipes under saturated conditions are influenced by the loading rate on the pipe and the drainage properties of soil. The current design guidelines such as ASCE (1984), ALA (2005) and DNV (2007) overlook the partially drained behaviour of sand, assuming it is fully drained. However, low permeable silty sand, which is abundant in offshore conditions, can induce partial drainage under high loading velocities. Thus, the development and the dissipation of excess pore pressure plays a significant role in determining the peak uplift resistance in saturated, low permeable sand.

The main objective of this study is to investigate the applicability of several finite element assumptions to reliably predict the uplift resistance of buried pipelines in the saturated and partially drained sand. The vertical soil-pipe interaction under large deformations is numerically simulated taking into account both geometric and material instabilities. A detailed parametric study is conducted on the effects of density, friction, dilation and loading velocity on the peak uplift resistance and the mobilised displacement of pipelines. It is shown that indeed partial drainage and loading rate have a significant impact on the predicted uplift resistance and therefore the results of this study propose directions for updating the existing design guidelines that ignore these effects.

RÉSUMÉ: La conception et la construction de pipelines sous-marins représentent une part importante des dépenses d'investissement en infrastructures offshore. Les principaux paramètres de conception requis pour la conception d'un pipeline contre le flambement par bouleversement sont la résistance au soulèvement maximale et le déplacement mobilisé correspondant. Ces facteurs sont influencés par le diamètre du tuyau et la profondeur d'enfouissement ainsi que par les caractéristiques du sol, telles que la résistance au frottement et la dilatation. Des études expérimentales récentes ont révélé que la résistance au soulèvement de tuyaux dans des conditions saturées est influencée par le taux de charge sur le tuyau et les propriétés de drainage du sol. Les directives actuelles en matière de conception, telles que ASCE (1984), ALA (2005) et DNV (2007) négligent le comportement du sable partiellement drainé, en supposant qu'il soit entièrement drainé. Cependant, le sable limoneux peu perméable, qui est abondant dans des conditions offshore, peut induire un drainage partiel sous des vitesses de chargement élevées. Ainsi, le développement et la dissipation de la pression interstitielle en excès jouent un rôle important dans la détermination de la résistance maximale au soulèvement dans le sable saturé peu perméable.

L'objectif principal de cette étude est d'étudier l'applicabilité de plusieurs hypothèses d'éléments finis pour prédire de manière fiable la résistance au soulèvement de canalisations enterrées dans le sable saturé et partiellement drainé. L'interaction verticale sol-tuyau sous fortes déformations est simulée numériquement en tenant compte des instabilités géométriques et matérielles. Une étude paramétrique détaillée est menée sur les effets de la densité, du frottement, de la dilatation et de la vitesse de chargement sur la résistance maximale au soulèvement et le déplacement mobilisé des conduites. Il a été démontré que le drainage partiel et le taux de charge ont un impact significatif sur la résistance au soulèvement prévue. Les résultats de cette étude proposent donc des instructions pour mettre à jour les directives de conception existantes qui ignorent ces effets.

Keywords: drainage effect; upheaval buckling; dense sand; large deformation

1 INTRODUCTION

Upheaval buckling is a major mode of failure during the service life of pipelines. Due to the high operating temperature, thermal expansion causes axial strains. When it is restrained by the friction of the soil pipe interface and end connections, axial stresses are generated, causing the buckling of the pipeline. Generally, the lateral buckling is prevented by high passive resistance. Hence, the weakest mode of failure is in the vertical direction which consequently leads to an exposure of the pipe to the surface or a bending failure. This is termed as upheaval buckling.

The uplift resistance provided by the backfill soil cover is the most significant design parameter of the buried pipeline design. If the pipe is simulated as a structural spring beam model, the soil stiffness, which involves both the peak resistance and corresponding displacement, is of paramount importance. These design parameters are governed by various geometrical properties such as the burial depth, diameter, surface roughness as well as soil properties like density, permeability, friction and dilation angle.

During the design of buried pipelines, it is common to assume either fully drained or fully undrained backfill. Generally, the current design practice assumes the drained condition for sand. However, the partial drainage can be evoked in the low permeable silty sand seabeds or when the pipe movement is relatively fast. Depending on the capability to dilate, the effective stress will be increased or decreased in the sand when the drainage is suppressed. Therefore, the degree of drainage dictated by the displacement rate and the permeability of sand have a substantial effect on the uplift pipe resistance.

2 LITERATURE REVIEW

The influence of the geometry (the pipe diameter and the depth) as well as the soil characteristics (density, friction and dilation angles) on the upliftresponse of a pipe has been well documented and they are taken into the account by current design guidelines (ALA, 2005),(*ASCE*, 1984),(DNV, 2007). However, the impact of the loading rate on pipelines buried in an offshore environment has only recently come to the light. Within this context, the influence of drainage on pipes buried in clay has been investigated recently. The accuracy of using undrained strength parameters and total stress analysis for the design of pipes is questioned (Tom & White, 2017).

Only a handful of studies have focused on the drainage effect on the uplift resistance of objects buried in the saturated sand. Vesic (1969) predicted that as the rate of loading increases, breakout loads of objects embedded in the ocean bottom may decrease if the sand density is below the critical and increase if it is above the critical.

Finnie (1993) investigated on loading rate effect on the pullout resistance of spudcans in the calcareous sand and reported the existence of a partially drained region. Similarly, Mangal (1999) experimentally explored the effect of the partial drainage of sand during the withdrawal of shallow footings. He deduced that the pullout stiffness increases with the displacement rate, hence a design based on the fully drained response is conservative.

Bransby and Ireland (2009), Byrne et al. (2013), Williams, Byrne, and Blakeborough (2013) experimentally studied pullout rate effects on the uplift resistance in saturated sand. Bransby and Ireland (2009) noticed that the increase in velocity resulted in a higher uplift load as well as the corresponding displacement in relatively dense backfills. Bransby and Ireland (2009) envisaged that different degrees of drainage can occur in the low permeable silty sand in seabeds. Tests performed in offshore conditions infer that practical pullout rates are large enough to trigger the partially drained condition in the relatively low permeable silty sand. Hence the assumption of the fully drained behaviour for sand, which is widely adopted by most of the design guidelines, is not accurate.

Byrne et al. (2013) and Williams et al. (2013) reported that faster rates reduced the uplift capacity of very loose saturated fine sand. They concluded that when uplift rates are high, there is no time for positive excess pore pressure to dissipate in the low permeable sand with high fine content, which resulted in a decrease in strength. Further, they noticed that the drainage effect is sensitive to the density of backfill. Even for loose sand, the contraction around the pipe caused a decrease in the uplift resistance whereas dilation in shear zones contributed to a strength increase.

With further investigation, Williams et al. (2013) scrutinised the transition relative density between the net contraction and dilation for a given embedment. Both Byrne et al. (2013) and Williams et al. (2013) advocated that it is not prudent to rely on the enhanced uplift capacities due to the rate dependence. Because, after the excess pore pressure has fully dissipated in the long term, the uplift resistance will return to the drained behaviour triggering the upheaval buckling.

All aforementioned studies confirm the significance of the degree of drainage on the mechanical response during the upheaval bucking of buried offshore pipelines. Evidently, its impact is different depending on the density of the backfill. Hardly any numerical investigation is conducted on this aspect. This study serves as a preliminary examination of the impact of loading rate, density, friction and dilation angles on the pipe uplift resistance. The accuracy of common constitutive models to capture the true mechanical response of upheaval bucking is scrutinised.

Table 1. Material properties of backfill

	Unit	**Loose**	**Dense**	**Medium dense**
Dry unit weight	kN/m^3	15.2	17.7	16.4
Poisson ratio		0.3	0.3	0.3
Elastic modulus	kPa	2500	3700	3000
Friction angle		32	44	35
Dilation angle		1	16	5
Cohesion	kPa	0.1	0.1	0.1
K_0		0.47	0.3	0.43
Void ratio		0.55	0.8	0.67
Consolidation coefficient		0.343	0.501	0.411

3 NUMERICAL PROCEDURE

An offshore pipeline buried in a sand backfill is the subject of this parametric study. The pipe is 50mm thick and 610mm in diameter. A moderate depth (H) to diameter (D) ratio of 4 is considered. The geometry and the position of the pipe are shown in Figure 1. Lateral boundaries are placed at 10m from the centre of the pipeline and the depth of the model is 5m which are sufficient to eliminate the boundary effect. A series of plane strain finite element analysis is conducted to examine the effect of loading rates, density, friction and dilation angles.

The soil-pipe interaction involves material, boundary and geometric non-linearities. The commercial FE software ABAQUS/Standard is utilised for the coupled soil-fluid analysis. The sand backfill is modelled as linear elastic-perfectly plastic with a Mohr-Coulomb yield criterion. The steel pipe is modelled as a linear elastic material. The material properties of soil are listed in Table 1. The permeability of pore fluid is 0.001ms^{-1}

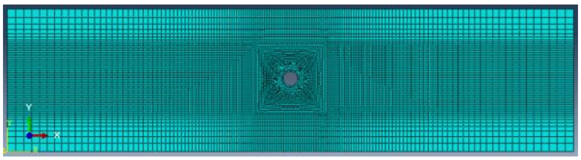

Figure 1 Geometry and mesh of the pipe model

3.1 Finite element mesh

Plane strain, biquadratic, reduced integration elements with eight nodes (Type CPE8R) are deployed for the analysis of dry sand. Since the effective stress analysis is employed in all saturated cases, the same element type with pore pressure (CPE8RP) is used for the coupled analysis. The pipe is modelled with the element type CPE8R for both cases. The sand domain is partitioned and a finer mesh is specified around the pipe as shown in Figure 1 with an average mesh size of 0.06m.

3.2 Contact between soil and pipe

The interaction between the pipe and soil is modelled with a surface based contact, which allows the soil to separate from the pipe. The Coulomb friction model is adopted to simulate soil-pipe interaction. The behaviour normal to the surface is simulated with a hard contact and the tangential response is modelled with a penalty contact. The interface friction angle is assumed to be half the soil friction. A surface to surface contact algorithm is prescribed, naming more rigid pipe as the master surface and the soil boundary as the slave surface.

3.3 Steps of Analysis

The analysis consists of three steps. During the initial stress condition, the bottom boundary is constrained in all three directions while lateral boundaries are fixed in the horizontal direction. For the saturated medium, a zero pore pressure is specified at the top surface indicating free drainage at the ground level. Additionally, for the coupled pore fluid analysis a linearly varying initial hydrostatic stress is specified, assuming the water table at the ground level. A uniform void ratio is designated as given in Table 1. A fully saturated condition is assumed throughout the analysis. Soil-pipe interaction is also introduced in this step.

During the geostatic step, the gravity of 9.81ms^{-1} is applied to the whole model to verify whether the initial geostatic stress is in the equilibrium with applied boundary conditions.

During the final step, prescribed values of displacement up to 0.3m are assigned vertically upwards to all the pipe nodes. Loading durations are specified in Table 2. The total force exerted on the pipe is taken as the sum of the vertical forces on all nodes of the pipe. The static general analysis is selected for the dry sand and the soil with transient consolidation is chosen for the pore fluid analysis. To take account of the geometric non-linearity in large deformation problems, the 'NLGEOM' option is activated.

4 RESULTS AND DISCUSSION

4.1 Force displacement curves

Figure 2 and 3 illustrate the total vertical reaction force exerted on the pipe against vertical displacement for dense and loose sand respectively. Dry as well as coupled analysis with different loading rates are explored. Arrows indicate the occurrence of the peak in the force displacement curve.

Naturally, during the initial equilibrium of pipe and soil, the buoyancy effect comes into the play. This phenomenon is known as floatation. In this study, the effect of floatation is eradicated by shifting the reaction force curve such that buoyancy equals to weight at initial equilibrium.

It is observed that the analysis prematurely ceased before reaching the final prescribed value of displacement for dry and saturated analysis with low velocities. In both figures, dry and saturated analysis with low velocities display a strain softening with a hint of reaching to a residual value.

In general, it is evident that the uplift resistance generally increases with the velocity of loading. Hence, the peak uplift force as well as the displacement required to reach the peak elevate with the loading rate on the pipe. When the loading velocity is raised beyond 0.3m/s, the uplift force on pipe keeps on increasing incessantly without reaching a plateau or reducing to a residual value. The reason underlying this phenomenon is the continuous increase in negative excess pore pressure in the dilative sand which leads to an elevation of effective stresses in the soil. Therefore, the rate effect is profound for dense sand and depends on the dilation angle. A similar pattern was observed for the numerical analysis of lateral soil pipe interaction in the saturated sand by Cheong (2006) , Soga, Pelecanos, Mallikarachchi, and Kumar (2015), Soga and Pelecanos (2014), Mallikarachchi, Pelecanos, and Soga (2018). This phenomenon which is termed as dilative hardening can be seen in undrained sand and rocks.

For the considered permeability of the sand, the rate of loading decides the degree of drainage and hence the effective stress conditions. When the loading rate is sufficiently small to dissipate all the generated excess pore pressure, the mechanical response must be very close to that of dry sand. However, in Figures 2 and 3, the dry force displacement curves are above the fully drained ones. This discrepancy is largely due to the difference in effective unit weights between the dry and saturated sand. It should be mentioned that a positive dilation angle is prescribed here even for loose sand. Otherwise, a decrease in effective stress should have been observed for higher loading rates in Figure 3.

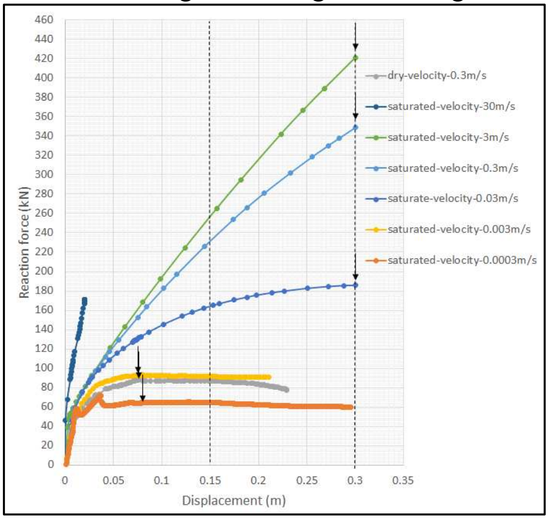

Figure 2. Force displacement relationships for different loading rates in dense sand

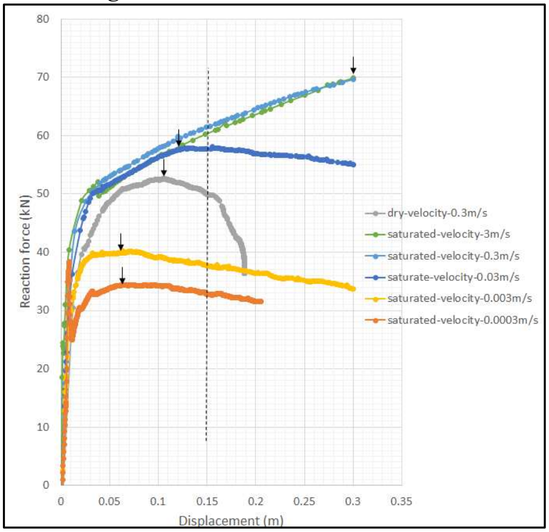

Figure 3. Force displacement relationships for different loading rates in loose sand

There can be several reasons for this observed analytical behaviour. The Mohr-Coulomb model used in this study incorporates constant dilation and friction angles ignoring the critical state concepts. Theoretically, dilation should be ceased to zero as plastic strains reach the critical state. Nevertheless, Cheong (2006) also observed a similar behaviour using the Nor Sand constitutive model which encloses critical state concepts. Therefore, it can be presumed that the progressive failure of consecutive soil particles around the pipe trigger the accumulation of pipe force.

4.2 Peak uplift resistance and corresponding displacement

Figures 4 and 5 display the normalised velocity vs normalised peak force and corresponding displacement respectively. As for greater velocities which do not show distinct peaks, the force at 0.3m displacement is taken as the maximum. Normalised peak force is defined as,

$$N_v = \frac{F_v}{\gamma' H D}$$

Where γ' is effective unit weight, H is the depth and D is the diameter of the pipe. The displacement is normalised by the diameter of the pipe. Normalised velocity is defined as,

$$V_n = \frac{vD}{c_v}$$

Where v is the uplift velocity and c_v is the coefficient of consolidation, which is a function of the permeability of the soil.

As normalisation eliminates the density effects, peak normalised forces of saturated sand at low velocities are close to those of dry sand. It is observed that the peak uplift forces increases with density, friction and dilation angle irrespective of the loading velocity. However, for saturated drained (at low velocities) and dry conditions, the influence of density is marginal. This is in line with the experimental observations on the dry sand by Trautmann and O'Rourke (1983).

When the loading velocity is increased in saturated sand, the effect of density becomes greater. Further, it can be observed that the peak uplift forces reach plateau closer to the two extreme velocities. Hence, the velocity range considered in this study can be divided into three regions depending on the degree of drainage: drained, partially drained and undrained. The boundary of these three regions seems to depend on density. The peak dimensionless forces of dense and medium dense sand show an increasing trend until the velocity is 3m/s. Hence, it can be presumed that the undrained region is beyond 3m/s for larger dilation angles. As for loose sand, the peak resistance reaches a constant when the velocity is greater than or equal to 0.3m/s. On the other hand, the drained condition occurs when the velocity is lower than 0.003m/s for all densities. However, these boundary velocities may be subjected to change for different H/D ratios.

The utilised constitutive model has a significant influence on these predictions. The constant dilation angle specified in Mohr-Coloumb model should be partly responsible for the continuous mobilisation of strength.

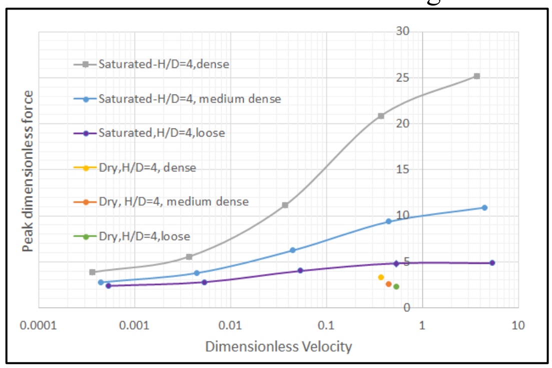

Figure 4. Relationship between normalised peak uplift forces against dimensionless velocity

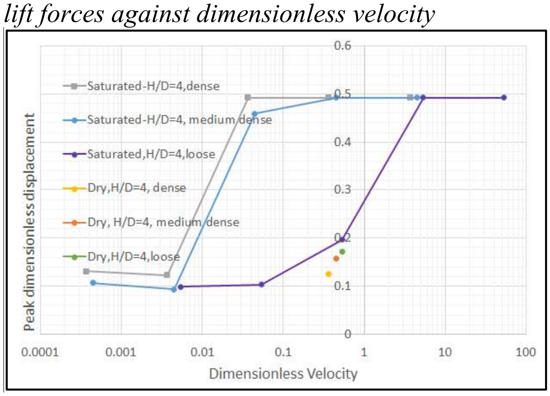

Figure 5. Relationship between normalised mobilised displacements at peak against dimensionless velocity

Normalised displacement at the peak of saturated drained dense sand is close to corresponding dry value. For loose and medium dry sand, the mobilised displacement is delayed than saturated drained one. Similar to the peak force, the influence of density on the mobilised displacement is greater when the velocity increases. This means a lower degree of drainage delays the mobilisation of peak force. When the velocity increases and the sand become partially drained, the influence of dilation is more pronounced with the development of negative excess pore pressure. The enhancement of strength in both dense and medium dense sand results in a delayed mobilisation of the peak.

Nevertheless, results in the undrained region in both Figures 4 and 5 are not reliable as distinct peaks or plateaus are not observed for velocities in that region.

4.3 Deformation mechanisms

The deformation mechanisms during upheaval buckling within three drainage regimes mentioned in the previous section are depicted in Figures 6 to 8. Only the dense backfill condition is shown here.

For dense dry and saturated drained cases, shear bands develop from the shoulder of the pipe and reach the surface as the uplift of pipe continues, showing a sliding block mechanism. This is consistent with experimental observations by White, Barefoot, and Bolton (2001). It is also observed that the slopes of this failure surfaces reduce with dilation angle. For loose and medium dense sand, deformation took a shape of a vertical slip failure combined with the flow around mechanism. It can be concluded that as density decreases, the mechanism changes from sliding block to flow around. Therefore, the surface heave is larger in denser sand. This is in line with experimental observations of Trautmann and O'Rourke (1983) for dry sand.

In the undrained region, shear bands emerge later compared to the dry and drained cases. The possible reason behind this can be the strengthening of sand due to the generation of negative excess pore pressure. Further, the number of tributary shear bands are greater for undrained cases. It can be postulated that as the velocity of loading increases, time for the dissipation of pore pressure reduces. Hence the number of tributary shear bands goes up decreasing the drainage distance between two bands to facilitate the dissipation of pore water more efficiently. Therefore instead of distinct macroscopic shear bands, an abroad spectrum of micro shear bands have taken place around the pipe. Generally, the void ratio inside these shear bands is greater than surrounding which makes them more drained. However, negative pore pressure outside these dilating shear bands enhance the effective stress and hence the uplift capacity.

Deformation patterns in the partially drained region, fall between the above two extremes. As the degree of drainage reduces, the number of tributary shear bands as well as their inclination (to the vertical) increases. Further, the maximum magnitude of plastic strain mobilised at similar displacements decreases. This suggests that further movement of the pipe is required to mobilise the full deformation mechanism, as the sand becomes more undrained.

Table 2. Variation of loading durations

	dry	saturated					
duration- s	1	0.01	0.1	1	10	100	1000

5 CONCLUSION

The pipe uplift resistance in saturated sand increases with loading velocity for all densities with positive dilation. The dimensionless peak uplift resistance as well as the displacement to reach the peak also rise with loading velocity. The faster the loading, the more displacement is required to deform the soil since the negative excess pore pressure increases the soil strength. Depending on the degree of drainage, the considered velocity range in this study can be divided into three separate regimes: drained, partially drained and undrained. The boundaries of these regions depend on the density of the backfill. The reduced drainage increases the number of shear bands and their inclination to the vertical. Peak dimensionless forces and corresponding displacements enhance with the density, friction and dilation. Finally, it is observed that the effect of these factors is greater when the loading rate is increased.

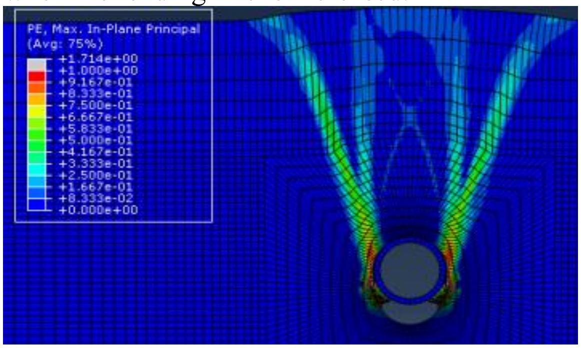

Figure 6. Plastic strain contours observed at 0.25 m displacement for dry dense sand

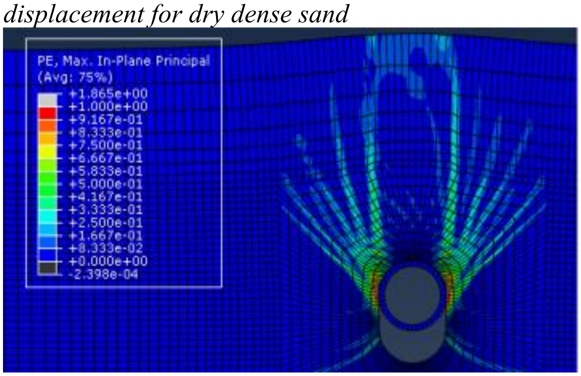

Figure 7. Plastic strain contours observed at 0.25 m displacement for saturated dense sand with loading velocity 0.3m/s

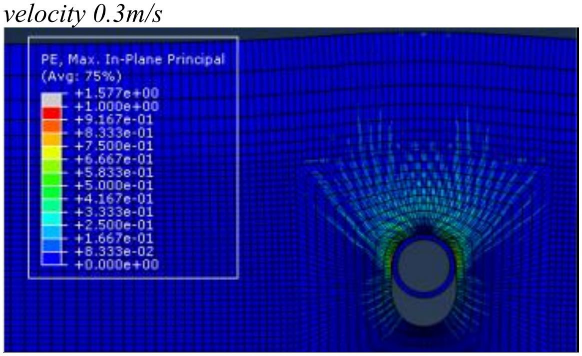

Figure 8. Plastic strain contours observed at 0.25 m displacement for saturated dense sand with loading velocity 3m/s

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