

# DISCUSSION OF LIMITING MUD PRESSURE DURING DIRECTIONAL DRILLING IN CLAYS

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## ABSTRACT

The Directional Drilling technique is now widely used internationally to install and replace underground conduits. Drilling mud is used during both the pilot drilling and pullback processes as a multifunctional medium. Mud loss and other damage can occur due to hydrofracture or blowout in the vicinity of the borehole as a result of high mud pressure. Current design methods, either simple rules of thumb or the Delft equation, are used to estimate maximum drilling mud pressure associated with shear failure, and these implicitly or explicitly assume that the initial ground stress condition is isotropic (i.e.  $K_0=1$ ). A new approach to estimate the limiting mud pressure is introduced in this paper which considers the effect of different initial geostatic stress conditions ( $K_0 \neq 1$ ). The limiting drilling mud pressure has been studied based on different ratios of shear strength to effective vertical stress ( $C_u/P'_0$ ) and different initial geostatic stress conditions ( $K_0$ ), and the results are compared with current industry methods. The studies indicated that soil type and initial ground stress condition control the mode of mud loss. Blowout is the dominant failure mechanism in normally consolidated (NC) and lightly overconsolidated (LOC) clay. Hydrofracture will be expected in heavily overconsolidated (HOC) clays where the coefficient of lateral earth pressure exceeds 1.8.

## RÉSUMÉ

La technique de forage directionnel horizontal est maintenant largement et internationalement répandue en pratique pour installer et remplacer les conduits souterrains. La boue de forage est utilisée durant le forage et l'élargissement du trou pilote comme milieu multifonctionnel. Dus à la pression élevée de la boue, la perte de boue et d'autres dommages peuvent se produire en raison de l'hydro fracture ou de l'éclatement aux alentours du trou. Des méthodes de calcul actuelles, que ce soit des principes de base simples ou l'équation de Delft, sont utilisées pour estimer la pression maximale de boue de forage liée à la rupture de cisaillement, et ces méthodes implicitement ou explicitement supposent que des conditions initiales des pressions dans le sol sont isotropiques (i.e.  $K_0=1$ ). Une nouvelle approche pour estimer la pression limite de boue est présentée dans cet article. Elle considère l'effet des conditions initiales des pressions géostatiques différentes ( $K_0 \neq 1$ ). La pression limite de boue de forage a été étudiée en utilisant différents rapports de résistance au cisaillement aux pressions verticales effectives ( $C_u/P'_0$ ) et aux différentes conditions initiales des pressions géostatiques du sol ( $K_0$ ). Les résultats sont comparés aux méthodes de calcul actuelles pratiquées par l'industrie. Ces études ont indiqué que le type de sol et des conditions initiales des pressions dans le sol contrôlent le mode de perte de boue. L'éclatement est le mécanisme dominant de rupture dans les argiles normalement consolidées (NC) et légèrement sur consolidées (LOC). L'hydro fracture est prévue dans les argiles fortement sur consolidées (HOC) où le coefficient des pressions latérales dans le sol excède 1.8.

## 1 INTRODUCTION

The horizontal directional drilling (HDD) industry has experienced great growth in the past decade. This method of installing pipelines and cables, to carry utility services under obstacles such as rivers, roads, canals, and railways was developed in the 1970's from technology used to deviate oil and gas well during drilling. To date, applications of this technology range across the civil engineering spectrum from the installation of utility conduits and natural gas pipelines, through municipal applications (i.e. water mains and gravity sewers), to environmental and geoconstruction applications such as the remediation of contaminated sites. The dominant attractions of directional drilling are minimized environmental disturbance, shorter construction schedules, as well as costs that may be reduced relative

to traditional pipelaying methods (i.e. cut and cover installation).

The directional drilling process includes three stages: pilot hole boring, pilot hole reaming, and pipe pullback (Allouche et al. 1998). Pressurized drilling fluid, a mixture of bentonite and water plus polymer additives, is generally used during the construction process to transport excavated drill cuttings back to the ground surface, to clean and cool the drill bit, to reduce sidewall resistance through the borehole during pullback, and to prevent the borehole from collapse (Ariaratnam et al. 2000). Mud loss problems caused by hydrofracture (tensile failure) or blowout (unconstrained plastic flow) of soil resulting from high mud pressure in the borehole is reported in many drilling projects each year (Christopher et al. 2003, Mathew et al. 2004, Harris, 2005, David et al.

2007). Mud loss prevention has become a critical issue which engineers and contractors face during project design and construction.

Since mud loss mostly occurs because of high mud pressure in the borehole, the study of maximum allowable mud pressure becomes a potential approach to control the mud loss phenomenon. Current design methods include expressions for maximum allowable mud pressure reported in the review by the US Army Corps of Engineers. However, this method was based on traditional cavity expansion theory assumed the soils are under isotropic initial ground stresses, so it does not consider anisotropic initial ground stresses where the initial horizontal stress in the ground is not equal to the vertical stress. A new approach is described here which considers different initial ground stresses conditions, and results are reported for allowable mud pressure in clays of different normalized strength (ratio of shear strength to effective vertical stress,  $C_u/P'_0$ ). This normalized cohesion and the relative magnitude of horizontal ground stress depend on the loading history of purely cohesive soils. Mud pressure estimates and failure mechanisms are explored for both normally consolidated (NC) and heavily overconsolidated (HOC) clays.

## 2 PAST STUDIES OF LIMITING MUD PRESSURE

In general, past numerical and experimental studies have identified two potential soil failure and mud loss mechanisms. The first mechanism is associated with generalized shear failure which generates unconfined plastic flow in the soil surrounding the borehole, so called "blowout", e.g. Arends (2003). Alternatively, tensile failure leads to open fractures permitting mud movement after minor principal stress becomes less than the tensile strength of the soil (normally conservatively assumed to be zero), Kennedy et al, (2004). This mechanism will be denoted "hydrofracture" here.

Various theoretical and experimental studies (Mori et al. 1987 and Panah et al. 1989) were performed to quantify the maximum allowable mud pressure in a vertical borehole. At No-Dig 2003, Arends introduced the closed form solution developed at the Delft University of Technology which represents the state of the art design procedure for estimating maximum allowable mud pressure reported by the US Army Corps of Engineers (Carlos et al. 2002). The closed form solution based on an examination of plastic zone development around the borehole, and the assumptions of this theoretical solution (called the 'Delft equation' subsequently in this article) can be summarized as follows: the borehole is axially symmetric, and the medium is homogeneous, isotropic, and of infinite size. The medium is assumed to be under an isotropic initial stress condition (i.e.  $K_0=1$ ), and response is elastic until the onset of shear failure, defined by the Mohr-Coulomb failure criterion as a function of cohesion and frictional angle. Increments of elastic deformation obey Hooke's law, elastic deformations in the plastic zone are neglected, and no volume change in the plastic zone is assumed

The maximum mud pressure is then expressed as:

$$P_{lim} = (P_f + c \cot \phi) \left\{ \left( \frac{R_0}{R_{p,max}} \right)^2 + Q \right\}^{\frac{-\sin \phi}{1+\sin \phi}} - c \cot \phi$$

where:

$P_{lim}$  Maximum allowable mud pressure

$P_f$  Mud pressure at onset of plastic failure

$$P_f = \sigma'_0 (1 + \sin \phi) + c(\cos \phi)$$

$\sigma'_0$  Initial effective stress

$\phi$  Internal friction angle

$c$  Cohesion of hosting soil

$Q$  A function of the shear modulus and effective stress

$$Q = \frac{\sigma'_0 (\sin \phi) + c(\cos \phi)}{G}$$

$G$  Shear modulus

$R_0$  Initial radius of the borehole

$R_{p,max}$  Maximum allowable radius of the plastic zone

For maximum radius of the plastic zone in real drilling projects, Arends(2003) suggests using a value of half of cover depth for purely cohesive soil and two-thirds of cover depth for frictional soils.

Kennedy et al. (2004a, 2004b) used finite element analysis to examine the response of an elastic-plastic soil during drilling, for a range of different mud pressures. They concluded that mud loss can occur after the initiation of a tensile fracture at the crown or springline of the borehole, the location depending on the initial ground stress conditions. Maximum mud pressure is estimated by determining the internal cavity pressure that brings tangential stress in the soil at the crown or springline of the borehole down to zero. The numerical results indicated that mud loss commences when the mud pressure in the borehole reaches:

$$P_{frac} = (3K_0 - 1)P_0 \text{ for } K_0 < 1,$$

and

$$P_{frac} = (3 - K_0)P_0 \text{ for } K_0 > 1$$

For cohesive soils where  $P_{frac} < 2C_u$ , tensile fracture is expected. Otherwise, blowout is expected before tensile fracture develops.

## 3 NEW APPROACH FOR LIMITING PRESSURE

Moore (2005) performed a series of finite element analyses to evaluate the efficiency of the Delft equation under different initial ground stress conditions. The results indicate that the Delft equation for blowout works

well if lateral earth pressure coefficient at rest  $K_0$  is equal to or close to unity (i.e.  $K_0 > 0.85$ ), but for lower coefficient, this solution gives an unconservative result. On the other hand, the Kennedy solution for tensile fracture (2004a, 2004b) cannot work for all practical lateral earth pressure coefficients  $K_0$ . Under some specific conditions, blowout will become the governing mud loss mechanism instead of hydrofracture. Therefore, a new approximate closed form solution (Xia and Moore, 2006) for blowout in purely cohesive material has been developed and is reported here. This can be used in those regions where lateral earth pressure coefficient  $K_0$  is not covered by either the Kennedy et al. (2004a, 2004b) or Delft (2003) solutions.

The new solution commences with consideration of elastic stress distribution around a borehole of initial diameter  $R_0$  in accordance with the Kirsch solution (Obert et al. 1967). Borehole pressures then increase to a value  $P_i$  and the borehole radius moves to  $R_i$ . The soil exhibits linear elastic response up to the limit of shear strength, as defined using the Mohr-Coulomb failure criterion. After that, it deforms with perfectly plastic behaviour. A plastic zone of radius  $R_p$  develops around the borehole and this plastic radius moves outward as borehole pressure increases further. No volume change ( $\Delta V = 0$ ) is expected in the plastic zone during the deformation for this purely cohesive material responding in an undrained condition.

Based on the assumptions outlined above, expressions can be derived giving the internal mud pressure as a function of the radius of the plastic zone (more details are provided by Xia and Moore, 2006):

$$P_i = C_u + \frac{1}{2}(3K_0 - 1)P_0 - C_u \ln \left[ \left( \frac{R_0}{R_p} \right)^2 + \left( \frac{C_u}{G} \right) \right]$$

for  $K_0 < 1$

$$P_i = C_u + \frac{1}{2}(3 - K_0)P_0 - C_u \ln \left[ \left( \frac{R_0}{R_p} \right)^2 + \left( \frac{C_u}{G} \right) \right]$$

for  $K_0 > 1$

where:

- $P_i$  internal mud pressure in the borehole
- $C_u$  undrained shear strength
- $K_0$  coefficient of lateral earth pressure at rest
- $P_0$  initial overburden earth pressure
- $R_0$  initial radius of borehole
- $R_p$  radius of plastic zone
- $G$  undrained shear modulus

Xia and Moore (2006) have evaluated the above solutions using finite element program AFENA (Carter 1992) by studying the development of the plastic zone (soil zone experiencing shear failure) around borehole. Figure 1 shows the development of the plastic zone around the borehole as the applied pressure increases for  $K_0 < 1$  (i.e.  $K_0 = 0.85$ ), and Figure 2 shows the radius of the zone of shear failure  $R_{p,max}$  normalized using the initial borehole radius  $R_0$ , versus the mud pressure  $P_i$  normalized using

the mean initial ground stress  $P_0$  at the crown of the borehole.

As Figure 1 indicates, mud pressure increases lead to growth of the plastic zone once initial yield has occurred in the soil. When the mud pressure is about 1.5 times the initial overburden pressure, shear failure initiates around the borehole. With further increases in mud pressure, the plastic zone grows steadily around the borehole. Up to mud pressure of  $3P_0$ , the maximum plastic radius occurs at a point directly over the crown. Once the mud pressure reaches  $3.5P_0$ , the point of shear failure which is furthest from the cavity deviates from above the crown, and accelerates towards the ground surface.

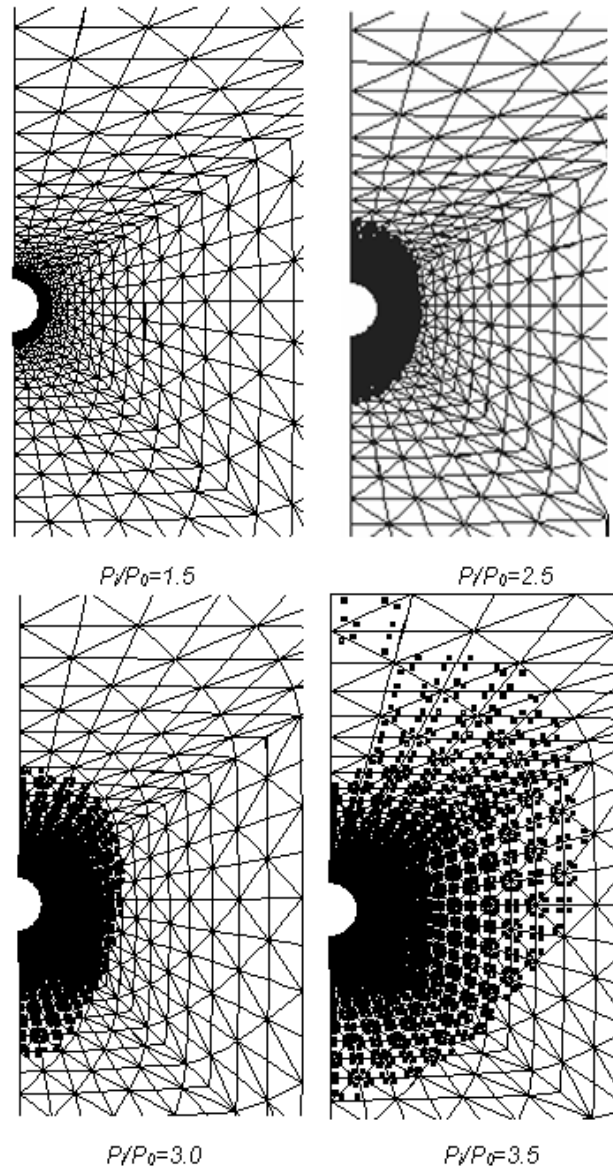


Fig.1. Plastic zone development with increasing mud pressures (Diameter of 0.4m, Cover Depth of 2m,  $K_0$  of 0.85,  $C_u$  of 20 kPa,  $\gamma$  of 20kN/m<sup>3</sup>)

Figure 2 demonstrates that the new approach works better than the Delft equation at lower load levels (where

the mud pressure is less than 80% of the limiting mud pressure), providing improved estimates of maximum plastic radius as a function of borehole mud pressure wherever the earth pressure coefficient  $K_0$  is less than unity. When  $K_0=1$ , the new procedure provides results identical to the Delft equation, and they both fit the finite element results well. For cases where  $K_0>1$ , the new procedure also avoids excessive (unsafe) estimates of the mud pressure that arise from the Delft equation once the plastic radius extends more than two-thirds of the distance towards the ground surface.

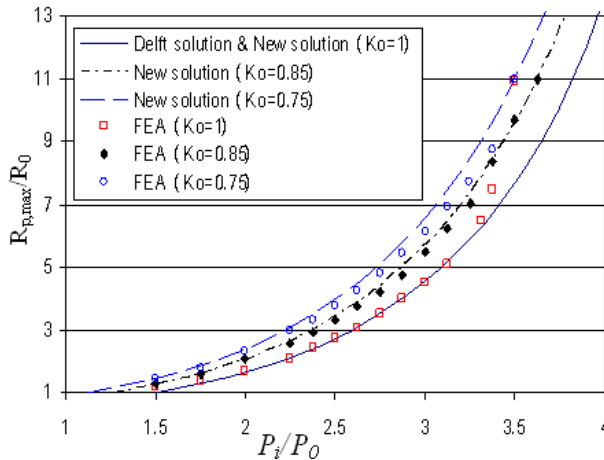


Fig.2. Comparison of extent of plastic zone calculated using the Delft Equation, the new solution, and finite element analyses for three different initial stresses conditions,  $K_0$  of 0.75, 0.85, 1 (diameter of 0.4m depth of 2m,  $\gamma$  of 20kN/m<sup>3</sup>)

#### 4 COMPARISONS WITH CURRENT DESIGN METHOD

As expressed above, the maximum allowable mud pressure and the controlling failure mechanism are dependent on the initial stress condition, the undrained soil shear strength, and the overburden pressure. Kennedy et al, (2004b) developed criteria for judging which type of failure criterion is critical, as depicted in Figure 3 for a given value of  $K_0$  and a specific strength ratio (i.e.  $C_u/P_0$ , the ratio of undrained shear strength to total vertical stress). As presented in Figure 3, soil elements at certain depths having specific undrained shear strengths and initial horizontal ground stresses defined by  $K_0$ , have mud loss mechanism dependent on the strength ratio defined by the blue line or the pink line. Hydrofracture is critical at strengths greater than those defined by these lines, and unconstrained shear failure (or blowout) is the mechanism controlling mud loss at lower shear strengths.

The strength ratio  $C_u/P_0$  for different type of soil materials has been well-documented based on numerous experimental studies. Ladd et al. (1971) reported that the strength ratio (i.e.  $C_u/P_0$ , the ratio of shear strength to effective vertical stress) is between 0.2 and 0.28 for normally consolidated (NC) clays, between 0.5 and 1.0 for lightly overconsolidated (LOC) clays, and may be

larger than 1.2 for heavily overconsolidated (HOC) clays. Assuming that the groundwater table is located at the ground surface and using typical values of soil unit weight, the values of  $C_u/P_0$  (total overburden stress) will be between 0.1 and 0.14 for NC materials, from 0.25 to 0.5 for lightly overconsolidated soils, and over 0.6 for heavily overconsolidated clay. Typical values of  $K_0$  for NC clay range from 0.5 to 0.6, for LOC clay the  $K_0$  value is close to unity, and for heavily overconsolidated clay,  $K_0$  can be 3 or more. It appears, therefore, that NC and LOC clays are most susceptible to blowout. Hydraulic fracturing is expected in HOC clay once  $K_0$  exceeds 1.8, while blowout is expected at lower  $K_0$  values.

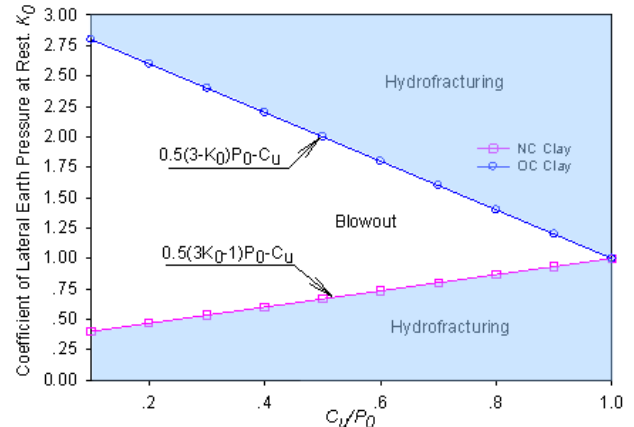
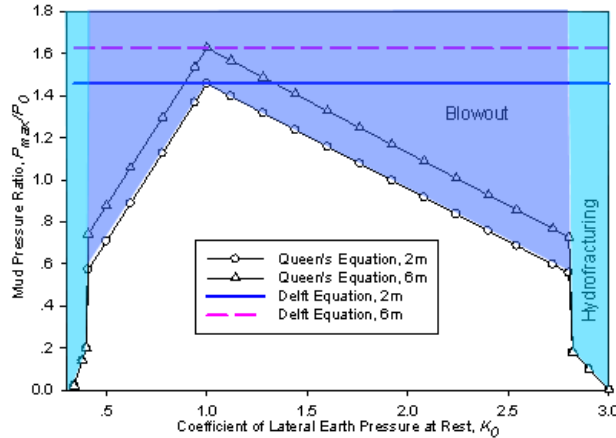


Fig.3. The criteria indicating the critical mud loss mechanism (hydrofracture or blowout).

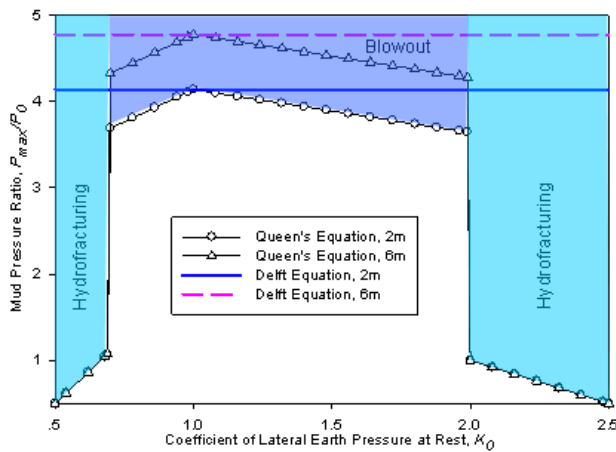
The allowable mud pressures were estimated based on the new approach under different initial ground stress conditions for NC and HOC clays, and also compared with the current design method (i.e. Delft equation) as plotted in the Figures 4 and 5. Two typical cover depths (i.e. 2m and 6 m) are considered. Figure 4 is the case of normally consolidated clay, assuming the ratio of the undrained shear strength to total overburden pressure is 0.1, and Figure 5 is for heavily overconsolidated clay which assumed the ratio of the undrained shear strength to total overburden pressure is 0.7. As illustrated, the current design solution which considers isotropic initial ground stress overestimates the allowable mud pressure except when  $K_0=1$ , where the two calculation methods generate the same maximum mud pressure. Obviously, the extent of overestimation increases as the  $K_0$  value decreases below  $K_0$ , and increases above  $K_0$ . The current design solution overestimates by about 1.5 times the overburden pressure for normally consolidated clay, and by around 0.6 times the overburden pressure for the heavily overconsolidated clay, using these specific soil properties. As this comparison indicates, the new equations may provide more reliable mud pressure limits to prevent mud loss during drilling project, since most ground conditions involve anisotropic stresses.

Numerous investigations (Duncan et al. 1976, Ladd, et al. 1977, Worth and Houlby 1985, Smith 1993 and Atkinson et al. 1995) have been undertaken to study the relations between undrained shear modulus and shear strength, and some studies also consider the effects of

overconsolidation ratio. Duncan and Buchigani (1976) suggested a value of  $G/S_u$  of 200 for normally consolidated clay, and this value was reasonably consistent with the data provided by Smith (1993) and Viggiani and Atkinson (1995); the value of  $G/S_u$  for overconsolidated clay can be estimated using the approach reported by Wroth and Houlsby (1985).



a) normally consolidated clay,  $C_u/P_0=0.1$



b) heavily overconsolidated clay,  $C_u/P_0=0.7$ .

Fig.4. Comparison of the pressure ratio between Delft equation and "Queen's equations" under different values of lateral earth pressure coefficient  $K_0$ ,

As concluded above, the NC clay is most susceptible to blowout; therefore, the following discussion of maximum allowable mud pressure will focus on these soft clay materials. Figure 6 illustrates the effects of borehole diameter and contribution of cover depth on the maximum allowable mud pressure for clays with specific soil input parameters (i.e.  $K_0$  of 0.85,  $C_u/P_0$  of 0.3). Three typical borehole diameters were studied (i.e. 0.2m, 0.6m, 1.0m). Under the so-called shallow cover depth (e.g. 4m given the borehole diameter is 0.2m), the maximum mud pressure ratio ( $P_{max}/P_0$ ) increases steadily with cover depth, however; if cover depth is larger than some critical cover depth, the so-called deep burial condition, the maximum mud pressure ratio approaches a constant value of 2.6 times the total overburden pressure. The same trend is exhibited for cases featuring other borehole

diameters, and the critical cover depth which distinguishes the shallow and deep burial conditions increases with increasing borehole diameter. Normalized burial depth  $H/D$  (i.e. Cover Depth divided by Borehole Diameter) can be used to define the demarcation of the shallow and deep burial cases, and a simple mud pressure chart could be generated to facilitate estimates of the maximum allowable mud pressure which could be many times the total overburden pressure.

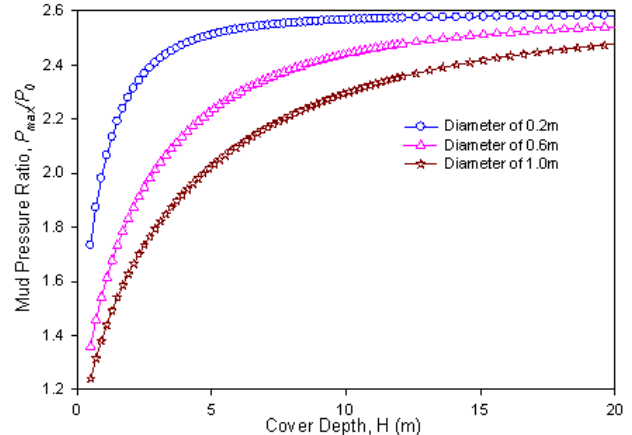


Fig.5. the contribution of the cover depth and borehole diameter to the mud pressure ratio for NC clay (i.e.  $C_u/P_0=0.3$ ,  $K_0=0.8$ ).

## 5 SUMMARY AND CONCLUSIONS

Control of maximum mud pressure to prevent mud loss during directional drilling is recognized as an important issue during project design and construction. Two mechanisms causing ground failure have been reviewed, namely the tensile failure and shear failure mechanisms. A new approach to estimate the maximum mud pressure has been reported here which explicitly considers anisotropic initial ground stress conditions. Finite element evaluations indicate that the new approach may provide a more reliable estimation than does the Delft equation. The new solution still neglects interaction between the borehole and the ground surface, and this eventually influences the solution when the plastic zone approaches the ground surface.

Parameters for use in these calculations should be estimated by experienced geotechnical engineers. Preliminary values for cohesive strength and the coefficient of lateral earth pressure at rest indicate that blowout is the dominant failure mechanism in normally consolidated and lightly overconsolidated clays. Tensile fracture is expected in heavily overconsolidated clays where the coefficient of lateral earth pressure exceeds 1.8. While the solutions presented here have been derived through careful consideration of the relevant soil behaviour, they are theoretical in nature, but it still did not consider other potential factors such as chemical interaction, fluid properties (i.e. fluid composition and temperature), also the initial pre-existing micro-fractures in the hosting soil around the periphery of the pressurized borehole. For practical application, engineering judgment should be used, or additional safety factors could be

considered to determine the maximum allowable pressure. Both field and laboratory studies would provide valuable guidance on the performance of these analytical methods.

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