Sliding resistance of offshore gravity foundations
La résistance des fondations gravitaires en mer coulissante

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ABSTRACT: For offshore gravity foundations on fine-grained soils, the sliding in the interface between the caisson and soil is often the design driver particularly if the intact soil is remoulded by excavation. This is exacerbated by the *H/V* < 0.4 criterion appearing in Eurocode 7 for undrained failure. To counteract the remoulding and to ensure a more favourable stress condition in the interface, a gravel bed is customarily placed between the excavated surface and the caisson base. The paper examines the combined interface conditions for drained and undrained failure in the light of the Code requirements for bridge and wind turbine foundations.

RÉSUMÉ : La conception de fondations gravitaires en mer est souvent gouvernée par calcul de résistance au glissement à l'interface entre le caisson et le sol, particulièrement lorsque les sols en place sont altérés mécaniquement par le dragage. Ceci est amplifié l'application du critère *H/V* < 0.4 figurant dans l'Eurocode 7 pour le cas non-drainé. Afin de diminuer cet effet et assurer un état de contrainte plus favorable à cette interface, une assise granulaire est typiquement mis en place en fond de fouille sur laquelle le caisson est ensuite installé. Ce papier examine les caractéristiques globales combinées de cette interface en conditions drainée et non-drainée dans l'esprit des exigences règlementaires pour les ponts et fondations d'éoliennes.

KEYWORDS: Gravel bed, gravity foundation, sliding resistance, clay till, failure conditions, Code of Practice

1. INTRODUCTION

For near and offshore windfarms and for strait crossing bridges gravity foundations are often the preferred solution. The big advantage is the savings in cost and programme time as the foundation caissons may be prefabricated independently of the ground preparations at the foundation site. To be competitive competent soil (possibly improved) is required at relatively shallow depth.

The advantage of large scale prefabricated reinforced concrete caissons as gravity foundations has been borne out by a large number of successful projects in Danish waters and abroad: the Storebælt bridges, the Øresund Bridge, the Izmit Bay Bridge, Nysted & Rødsand Windfarms, and Thornton windfarm just to mention a few (e.g. Figure 1 to Figure 3).

Figure 1. The Storebælt bridges, Denmark; (a) West Bridge in front & East Bridge at the back; (b) Prefab. yard for East Bridge caissons.

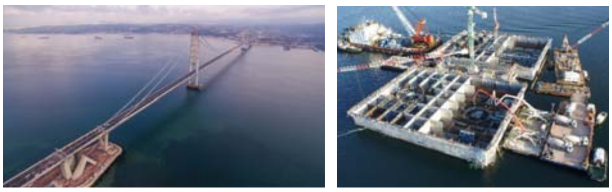

Figure 2. The Izmit Bay Bridge, Turkey; (a) overview; (b) Tower foundation caisson

Figure 3. Offshore windfarms (a) Thornton (Belgium); (b) Nysted (Denmark); (c) Rødsand 2 (Denmark)

In order to facilitate transfer of vertical and horizontal stresses from the concrete caisson to the underlying soil a gravel bed is customarily prepared. The functional requirements are:

(i) To safeguard against unwarranted stress concentrations in the gravel bed concrete interface (avoid hard points)
(ii) To ensure that a potential sliding surface is at concrete/gravel, gravel/gravel or gravel/intact soil interface
(iii) To ensure filter stability between gravel and soil and hydraulic stability during caisson lowering

The interface between the excavation and the gravel bed is critical if constituted by a remoulded zone of the underlying soil. This has often been a key point in the validation and certification of the foundations. Likewise, the evenness of the gravel bed and the interface to the concrete base of the caisson are critical issues.

For the projects in Denmark, clay till is often the predominant soil allowing for a caisson gravity solution but also challenging the concept of undrained sliding resistance as the Eurocode demands that *H/V* < 0.4 for sliding in undrained conditions in fine grained soil (clay).

This paper re-evaluates design assumptions for sliding in the light of the evidences found in the literature and the theoretical interrelationship between bearing capacity failure, sliding resistance and properties of the gravel bed and undisturbed and remoulded clay till.

2. UNDRAINED BEARING CAPACITY – INTACT SOIL

2.1 *General*

The bearing capacity formula, for undrained failure on a soil with constant undrained shear strength c_u, is well documented both theoretically and in practice (see e.g. Steenfelt, 2003)

$$\begin{aligned}
&\frac{Q}{A'} = c_u N_c^0 s_c^0 i_c^0 + q' \\
&N_c^0 = \pi + 2 \\
&s_c^0 = 1 + 0.2\frac{b'}{l'} \\
&i_c^0 = 0.50 + 0.5\sqrt{1 - \frac{H}{A' c_u}}
\end{aligned} \qquad (1)$$

To correctly deal with eccentric loading (where an unloaded fissure between footing and soil surface may form) the load Q and the surcharge q are formally reduced by the pore pressure at foundation level outside the footing, and hence Q' and q' in EQ. (1). The effective footing dimensions are width b' and length l.

To simplify the problem without loss of generality Eq. (1) may be expressed for a centrally loaded strip footing on an unloaded horizontal surface as ($q'= 0$ and $b'/l'= 0$, $s_c^0 =1$):

$$\frac{Q}{c_u A'} = N_c^0 i_c^0 = \frac{\pi}{2} + 1 + 2m_1 + \sin 2m_1; \quad \cos 2m_1 = \frac{H}{c_u A'} \qquad (2)$$

EQ. (2) provides the mathematically correct upper limitation in terms of bearing capacity for the combination of horizontal and vertical loads, H, V (≤Q') as shown graphically in Figure 4.

The graph demonstrates a number of interesting points. For $H= 0$, the maximum possible load V is obtained for the maximum size of the rupture figure (depth $b'/2$ and sqrt(2)$b'/2$ below the mid-point and edge, respectively). As H increases, the bearing capacity decreases until the rupture figure degenerates to a line below the footing, i.e. with zero depth. This coincides with sliding in the soil/footing interface. At this point, $V_0 = (\pi/2+1)A'c_u$ and the ratio $H/V_0 = 0.389$, which is seemingly the rationale for the additional requirement to Eq. (1) in the Danish Code of Practice (DS415, 1998) that $H/V < 0.4$.

The expression is valid for both characteristic and design values. Implicitly, the criterion corresponds to an increase in the partial factor of safety, γ_c, proportionally with V_0/V.

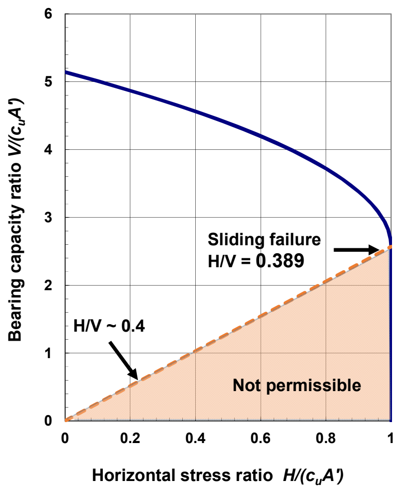

Figure 4. Non-dimensional bearing capacity graph for strip footing loaded by V, H

2.2 *The infamous H/V < 0.4 criterion*

The $H/V < 0.4$ criterion was introduced and argued by Mortensen (1983) as an example of limit state design insufficiency related to the anchor block design for the Storebælt East Bridge (see Figure 5).

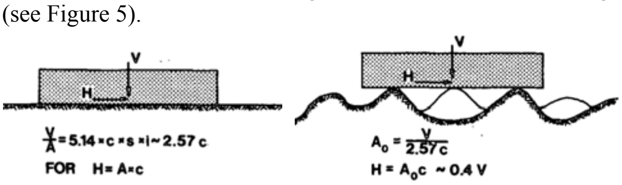

Figure 5. Inclined and centrally loaded strip footing on clay till (After Figure 10 from Mortensen (1983).

Subsequently, this was adopted as a design criterion in both Eurocode 7 (DS/EN 1997-1, 2015) and DNV (2004). In the Author's opinion, it is a conservative requirement without theoretical or testing back up. Notably, the H/V criterion is not included in DNV (2014).

The argument was, that for a "light" foundation with $V/A < 2.57\ c_u$, only part of the foundation, $A_0 = V/(2.57\ c_u)$, would be in contact with the clay till during sliding, due to unavoidable irregularities at the surface. Hence, the permissible horizontal load reduces to $H = A_0 c_u = V/(2.57) \sim 0.4\ V$ (based on the extreme interface condition of Figure 5).

This has the consequence, that undrained sliding would only be the decisive load scenario if the effective friction angle, φ', of the clay till were lower than arctan(0.4) = 21.8° in contrast to the values for Storebælt clay till in the range 33 - 34° (corresponding to $H/V = 0.65 – 0.67$). In the development of the argument it seems forgotten that undrained failure is only a theoretical concept, no matter how convenient it may seem.

The underlying soil behaviour is still dictated by the effective strength parameters φ', c'. These are mathematically determined as tangent parameters to the Mohr-Coulomb curved failure condition.

Using a straight line approximation for the curved failure envelope with $c' = 20$ kPa, $\varphi' = 34°$ and $c_u = 200$ kPa the consequence of the Code requirement H/V < 0.4 (equivalent to $c' = 0$ kPa, $\varphi' = 21.8°$) is demonstrated in
Figure 6.

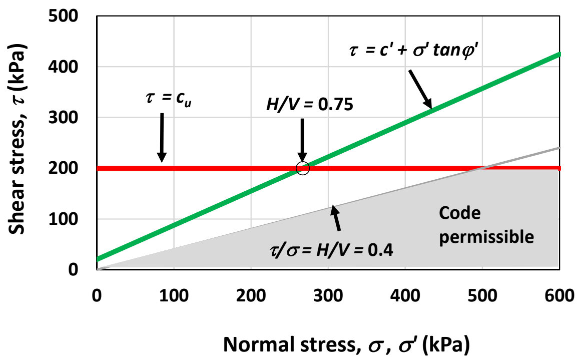

Figure 6. Mohr Coulomb failure criteria for total and effective stresses compared with undrained Code demand

For undrained conditions during the sliding process (e.g. constant volume), the upper bound would be $\tau = c_u$. However, if drainage is occurring in the very thin sliding zone then the upper limit of the shearing resistance would be $\tau = \min\{c' + \sigma' \tan\varphi', c_u\}$. This also explains the fallacy in the arguments for $H/V < 0.4$ in Figure 5b, as a possible reduced contact with the clay asperities from the consolidation phase would demand changes in the effective stresses during sliding and hence, violate the assumed undrained (volume constant) assumption.

Even if only spurious contact between the concrete and the clay till surface was established during the initial consolidation phase, the effective stress in the interface would not change as any change in total stress would be counteracted by changes in pore pressure, which is independent of contact area.

The introduction of a gravel bed between the concrete caisson and the underlying in situ clay till changes the interface conditions radically. At the onset of sliding, the very permeable gravel bed material easily accommodates dissipation of pore pressure changes (required to maintain the undrained failure). This means that the failure condition for the clay till, in terms of effective stresses, will dictate the maximum shear stress for $H/V > 0.4$. For the example in Figure 6, the transition from undrained to drained failure would be at $H/V = 0.75$.

Figure 7 illustrates the complexity of the sliding question comparing the theoretical drained and undrained bearing capacity of strip footings on intact clay till with realistic parameters and different values of the surcharge, q' next to the footing. It is obvious that the $H/V < 0.4$ criterion is conservative, and that the sliding/resistance criterion may be activated in drained or undrained conditions depending on the geometry and the soil parameters.

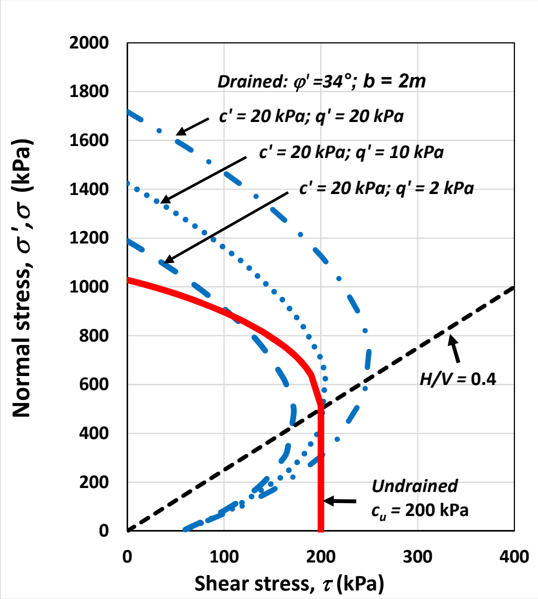

Figure 7. Drained and undrained bearing capacity for strip footings on intact clay till: φ' = 34°, c', q' & c_u = 200 kPa

2.3 *Inferences from bearing capacity theory*

As seen in Figure 4 the point of sliding failure coincides with the point where the ratio of undrained shear strength and vertical total stress, c_u/σ_v, equals 0.389 ≈ 0.4. This ratio is the coincidentally the same as the c/p ratio assumed for the Storebælt and Rødsand clay tills in the normally consolidated state, i.e. $(c_u/\sigma'_v)_{nc} \sim 0.40$.

This may explain some of the myths associated with the number 0.4, as it expresses the ratio associated with bearing capacity at sliding failure as well as undrained shear strength ratio for normally consolidated condition for clay till.

Yet another interesting feature, illustrated by Figure 4, is that at the maximum vertical bearing capacity the ratio $c_u/\sigma_v = 0.2$. Thus, if any vertical load exceeding the load corresponding to sliding failure, is maintained long enough for consolidation to take place, the undrained shear strength in the footing interface will increase irrespective of the current overconsolidation ratio $OCR = \sigma'_{v,pc}/\sigma'_{v0}$, where σ'_{v0} here is the current vertical effective stress below the footing

The last implicit feature of the bearing capacity formulation is that at $H/V = 0.389 \sim 0.4$, the soil in the sliding interface will reach a value of $OCR = 1$, if the load was maintained to allow the pore pressures to dissipate.

This may be seen using the SHANSEP formulation for the undrained shear strength ratio (for clay till at Rødsand):

$$\left(\frac{c_u}{\sigma'_{v0}}\right)_{pc} = \left(\frac{c_u}{\sigma'_v}\right)_{nc} OCR^{\Lambda} = 0.4\left(\frac{\sigma'_{pc}}{\sigma'_{v0}}\right)^{0.85} \quad (3)$$

It follows directly that $c_u/\sigma'_v = 0.40$ requires $OCR = 1$.

3. REMOULDED SOIL/FOOTING INTERFACE

In case a thin layer of completely remoulded clay (lower undrained shear strength) is present in the interface between the footing and the underlying intact soil the bearing capacity calculations change. For the layer to be representative of the interface strength the thickness must exceed the amplitude of unevenness of the excavated surface.

When a remoulded layer of thickness, z_{rem} ($< b'/2$), is overlying the top of the intact clay till surface the bearing capacity factor changes to (Poulsen, 1969):

$$(N_c^0 i_c^0)_{squeeze} = 1 + \pi - 2\left[\frac{H}{A c_u}\right]^3 + \left(1 - 0.8\frac{H}{A c_u}\right)\frac{b'}{2 z_{rem}} \quad (4)$$

The significant increase in the bearing capacity factor for reducing thickness of the remoulded layer is counteracted by the decreased undrained shear strength of the remoulded layer, especially for inclined loads, and hence the undrained shear strength of this layer would be decisive for the sliding capacity.
The influence of a remoulded layer on the sliding capacity of gravity base foundations was studied extensively in connection with the design of the foundations for the Storebælt Fixed Link (e.g. Steenfelt, 1993 (East); Hansen et al., 1991 (West)).

4. EVIDENCE FROM STOREBÆLT LARGE SCALE SLIDING TESTS

In Hansen et al. (1991) large scale tests on concrete blocks, for the West Bridge (WB), are described (1 by 2 m^2; H acting perpendicular to the short side). The tests were carried out as displacement controlled tests with a displacement rate of 26 mm/sec simulating the effect from ship impact. The results in terms of the ratio H/V_c at failure (V_c is the consolidation load, subsequently maintained) were:

$$H/V_c = \tau/\sigma'_c = \begin{cases} 0.53 & \text{(intact)} \\ 0.46 & \text{(disturbed)} \\ 0.42 & \text{(remoulded)} \end{cases} \quad (5)$$

where σ'_c is the vertical effective consolidation stress applied and maintained throughout the test. In terms of bearing capacity of the concrete test blocks, this corresponds to starting at a vertical stress well below the stress corresponding to the bearing capacity of a vertically loaded foundation as seen in Figure 8. The tests demonstrate that a value of $V/(c_u A') < (\pi+2)/2$ is in fact possible for sliding with $H/(c_u A') = 1$, i.e. corresponding to $H/V > 0.4$ at failure. The "cut-off at higher V-values corresponds to failure in a direction perpendicular to the long side of the foundation.

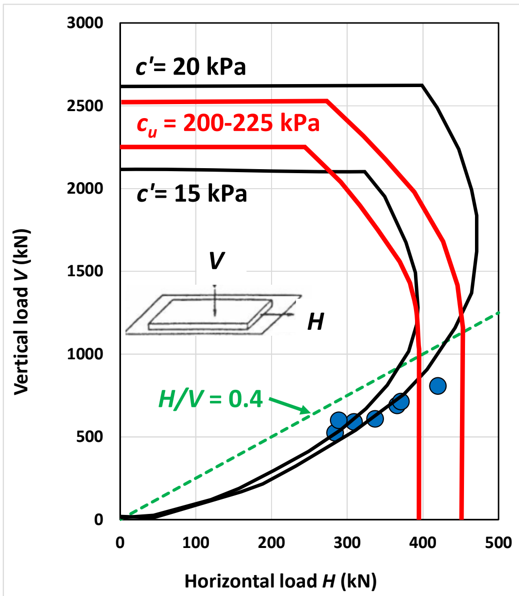

Figure 8. Large scale sliding tests with 1 by 2 m² concrete foundations on intact clay till for West and East Bridges Storebælt (Steenfelt (1991)

The three tests for the East Bridge on intact clay till with displacement rates of 0.030 and 0.0021 and 0.0017 mm/sec resulted in H/V = 0.52, 0.53 and 0.48, respectively.
In the tests on remoulded clay, the thickness of the remoulded layer was about 100 mm, corresponding to $b'/(2\ z_{rem})$ = 10. The corresponding bearing capacity at sliding is then $V/(c_u A')$ = 4.2 (Eq. (4)) compared to 2.57 for intact clay. Thus, the remoulding strength, $c_{u,rem}$, may be as low as 4.14/2.57 = 0.61 $c_{u,intasct}$, without reduction in sliding resistance

In the field sliding tests, the test conditions varied considerably (consolidation stress, thickness of bedding sand and disturbed/remoulded layers, displacement rate and more) and hence it is difficult to obtain a very clear picture of the impact of the individual factors.

However, the tests are simulating the behaviour of a foundation, albeit at model scale (see

Figure 8). The test results for sliding on intact clay (with 100 mm sand bed) are compared with the theoretical bearing capacity calculations for the rectangular footing for drained as well as undrained conditions using the range of parameters for the clay till at the site. The figure illustrates that failure largely is dictated by the effective strength parameters of the clay till and that the H/V < 0.4 criterion is not appropriate.

5. REALISTIC INTERFACE CONDITIONS

At Rødsand 2 the intact clay till at foundation level had the strength parameters c_u, = 250 kPa (based on an assumed cone factor N_k = 15) and φ'_{tr} = 33°, c' = 25 kPa based on triaxial tests on 70 by 70 mm specimens with smooth pressure heads.

The foundation level was achieved by excavation using a backhoe. The shovel width was about 1 m and to facilitate excavation in the very hard clay till it was fitted with "teeth" about 0.15 m long. The excavation was carried out radially from the periphery of the circular foundation pit towards the centre. The tolerance was ±0.2 m around the nominal foundation level. After excavation, the foundation pit was cleaned for soft material using an air lift guided by diving assistance. From the experience from Nysted this will result in an uneven surface with only tens of millimetres of softer material in pockets on top of the intact clay till. All lumps of clay till, liberated as part of the excavation process was removed by the air lift. The condition of the surface, was subsequently verified by video sweeping, "penetration testing" executed by the divers and by five CPT tests immediately before placing the gravel bed, the top of which is 0.5 m above the nominal excavation level. Figure 9 shows a principal sketch of the interface.

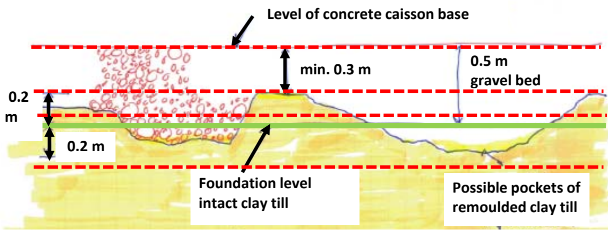

Figure 9. Principle of interface between clay till and gravel bed. The broken lines indicate sliding failure surfaces in (from top): (i) concrete/gravel inter-face; (ii) entirely in gravel; (iii) 50% in gravel 50% in intact clay till; (iv) intact clay till

Small amounts of settling suspended sediments and local pockets of remoulded/disturbed clay till was penetrated by the gravel layer. The gravel had a mean grain size about 20 mm and maximum grain size 90 mm corresponding to the gravel material applied for the Storebælt Bridges. It consisted of Hyperite with a high compressive strength (σ_c ~170 MPa; ρ_s ~3.12 Mg/m³), and even when placed in a loose state, the triaxial angle of friction is very high (φ'_{tr} > 50°), see Steenfelt & Foged (1994).

Using the methodology described, the certifying body accepted, that sliding would occur only in the interface options indicated in Figure 9, and hence the issue of the remoulded layer was resolved; but at the time the H/V < 0.4 was still active and needed to be considered. However, due to the very high partial factor on the undrained shear strength, γ_c = 1.8, and high overturning moments, very reduced effective area, A', the most critical criterion turned out to be, $H/(c_{u,char} A')$ < 1.8.

4. CONCLUSION

The interface between gravity foundations (concrete caissons) and the underlying fined grained soil (here clay till) has been examined. In accordance with Code requirements, it is always necessary to check the resistance of the foundation for both undrained and drained situations. Hence, the requirement H/V < 0.4 for undrained conditions is superfluous, as demonstrated by theoretical considerations, and backed up by testing evidence.

The customary application of a gravel bed between the caisson and the underlying soil promotes drained condition in the gravel/soil interface. The risk of sliding on a remoulded layer is at the same time eliminated if proper attention is paid to the preparation of the interface (compaction forcing gravel into and through the remoulded zone or removal of all remoulded material combined with an uneven surface of the intact soil.

5. ACKNOWLEDGEMENTS

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