Partially Mobile Subsea Shallow Foundations – A Practical Analysis Framework

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Abstract

The geotechnical design of partially mobile subsea foundations (i.e., mudmats) for pipeline/flowline end terminals (PLETs) is presented in this paper. A partially mobile mudmat represents a fit-for-purpose engineering solution that has significant commercial competitiveness. It lies between a fully anchored mudmat (which is designed for negligible movements but may be too large causing installation issues or needs corner piles to anchor) and a fully mobile mudmat (which moves to fully accommodate the expansion of the connected pipeline, but may suffer excessive settlements that compromise the structural integrity), and is suited to deepwater soft soil conditions. The motivations of the paper are to help to mature this new concept and technology for practical design and to inspire future research to improve the accuracy of predictions. The objective of the paper is to present simple new analytical solutions to predict the long-term accumulated displacements and rotations of a partially mobile mudmat on soft clayey deposits subjected to cyclic loading. The proposed displacement prediction framework combines established elements of consolidation theory, plasticity theory, and Critical State Soil Mechanics (CSSM). Typical ranges of soil properties pertinent to a partially mobile mudmat are provided for the deepwater Gulf of Mexico (GoM) soft clays, and a design analysis example is provided. For these conditions, it is concluded that the dominant displacements of a partially mobile mudmat are caused by primary consolidation and plastic failure. Recommendations for further improvement are listed to inspire further research.

Key words: Partially Mobile, Mudmat, Critical State, Consolidation, Cyclic, Clay

Introduction

Rectangular subsea mudmats with peripheral short skirts (typically around 0.3 meters in length) are commonly used as the foundations to support subsea facilities, e.g., a pipeline end termination structure (PLET). The design of subsea mudmats usually follows the conventional factor of safety or partial action/material factor concepts. For example, the American Petroleum Institute (API) guideline (API RP2GEO, 2014) adopts the global factor of safety approach while the ISO guideline (ISO 19901-4, 2016) recommends using the factored load and factored resistance approach. In essence, the underlying philosophy of the conventional design approach for mudmats is to ensure the resistance of a mudmat is greater than the imposed loads, which include but are not limited to pipeline (in-field flowlines and export pipelines are both termed pipelines in this paper for simplicity) thermal expansion loads, jumper loads, subsea equipment loads, connection misalignment loads etc.. Allowing a certain safety margin, the design analysis ensures the mudmat will undergo only pre-failure movements under all design conditions and will remain stable.

The oil and gas industry now is moving to deep and ultra-deep water areas, which can be approximately defined as water depths greater than about 600 m or 2000 feet. In deepwater areas, pipelines usually carry high pressure and high temperature (HPHT) fluids, and soils are much softer with water contents typically around 150% at the seafloor. In these conditions, the conventional design approach becomes increasingly uneconomical, because the dimensions of a mudmat can prevent it to be installed with a pipeline lay vessel, or the mudmat must be fitted with extensions after installation in order for it to fit into the workspace of the vessel. Alternatively, hybrid foundations with pin-piles installed at the corners of a mudmat may be considered (Dimmock *et al.*, 2013); however, additional cost and risk exposure is induced by additional offshore installations.

Recent innovations in offshore geotechnics lead to the design of performance-based fully mobile subsea mudmats that can directly move back and forth with the pipeline thermal expansions and contractions. Significant progress has been made in this area in the past decade: centrifuge tests (Cocjin *et al.*, 2014; Wallerand *et al.*, 2015), numerical simulations and laboratory soil element tests (Deeks *et al.*, 2014; Feng and Gourvenec, 2016), and analytical frameworks by Cocjin *et al.* (2017) and Corti *et al.* (2017) have been presented to advance this concept. The major benefit of a fully mobile mudmat is that the size can be much smaller than it has to be based on the conventional design approach that aims for a fixed foundation. The technical justification for a smaller and fully mobile mudmat is that the mudmat does not need to resist to the imposed load, but instead moves to relieve the load. The design criteria then become to ensure that the movements (i.e., horizontal movement, vertical settlement, and rotation) of the mudmat are within the subsea structure tolerance. For mudmats with rigid jumpers, these tolerances are typically around half a meter in the vertical direction and one to two degrees for the rotation. The tolerance for the horizontal movement depends heavily on the subsea layout but typically is more than the tolerance in the vertical direction by up to several meters. Therefore, these tolerances are more than an order of magnitude greater than that for buildings or fixed offshore platforms (typically a few inches or less). All the above studies focused on a mobile mudmat that is rigidly connected to the pipeline and moves back and forth with hundreds of expansion-contraction cycles during the design life. The horizontal cyclic amplitude (i.e., the peak or the trough to the neutral position) is typically in the range of 0.5 m to 2 m in each cycle, which corresponds to a full shut-in and start-up cycle of the pipeline.

An intermediate design between a conventionally designed mudmat and a fully mobile mudmat is a partially mobile mudmat with a sliding mechanism (slider) in the PLET. This concept of a partially mobile mudmat is similar to a fully mobile mudmat but takes the advantage of the slider that absorbs the majority of the pipeline expansion and contraction. Therefore, the mudmat can only be dragged by the pipeline when the expansion or contraction meets the end of the slider, and subsequently the horizontal cyclic movement amplitude will be much smaller (typically less than 150 mm)), compared to the total movement of the pipe end. This partially mobile mudmat on soft clayey deposits is the focus of this paper, with the motivation to mature this new technology for practical design and to inspire future research. The current study adopts the same analytical framework as Cocjin *et al.* (2017) for a fully mobile mudmat that is based on Critical State Soil Mechanics (CSSM) (Schofield and Wroth, 1968), with the added features: (i) the smaller sliding distance of a partially mobile mudmat is considered; (ii) a method for assessing the accumulated rotation is provided, so the full range of tolerance checks can be addressed; (iii) the analysis framework is simplified to closed-form analytical solutions for engineering practice, and (iv) properties of typical normally consolidated and lightly overconsolidated clays in the deepwater Gulf of Mexico (GoM) are provided to aid design practice.

Partially Mobile Mudmat Design Task

The problem addressed in this study is depicted in Figure 1. As shown, a rectangular surface mudmat with a width of B and a length of L rests on a half-space clayey deposit and is subjected to in-plane loading and displacement. The vertical load V has an eccentricity e_y relative to the origin, *O*, the geometric center of the mudmat in the y-axis, and a horizontal cyclic sliding displacement h_y occurs along the y-axis acting h_0 above the mudmat surface through a pivot which is free to rotate without inducing moment loads at the pivot. Therefore, the design problem is simplified into an in-plane loading problem. The design task is to determine the long-term accumulated total vertical settlement δ_v and total rotation θ_x about the x-axis from hundreds of horizontal loading cycles.

The driving factor to cause the sliding of the mudmat considered here is from the pipeline shut-in when the contraction of the pipeline exceeds the length of the slider in the PLET. This leads to contact between the end of the slider and the pipeline, which drags the mudmat sideways. However, during start-up, the expansion of the pipeline will be absorbed by the slider, the direction of the horizontal load is reversed and only a reduced value is mobilized, equal to any friction transferred through the slider. Thus, the slider is a 'weak link' to absorb the expansion, while the mudmat itself remains in the same position during the expansion of the pipeline. A similar situation may apply at the opposite end of the pipeline, but with the expansion phase causing the slider limit to be reached leading to mudmat movements.

Therefore, the scenario considered in this study can be simplified to be one-way displacement-controlled cyclic loading along the y-axis with a displacement amplitude of h_y in each cycle. The horizontal load, H_y, resulted from the horizontal sliding, is not known as a prior but equals to the sliding resistance that is determined by the clay properties from shearing and consolidation, and the associated failure mechanism. For the same reason, the overturning moment M_x about the center of the foundation is not known but is determined by H_y and the vertical load, V, and the eccentricities of these two loads, h_0 and e_y.

In reality, a mudmat is usually subjected to six-degree of freedom loading due to other minor attached components, such as jumpers, and also has short skirts. However, for the case when a mobile mudmat is proposed, the major component of loading is from the cyclic sliding displacement h_y, so the effect of any horizontal force in the direction of the x-axis is small and is neglected in this study. For the same reason, the overturning moment about the y-axis and any torsional loading is also neglected. In addition, this study simplistically focuses on a surface mudmat from the perception that the skirt has relatively minor effect because the passive resistance acting on the soil is expected to be pushed aside and then left away from the foundation in a berm in a displacement-controlled cyclic loading, so makes no further contribution to the sliding resistance. The mudmat then slides on a soil plane at skirt tip level, and this can be simplified to a surface foundation at a greater eccentricity, e_v.

Analysis Framework

Given the design task of determining the long-term vertical settlement and rotation from cyclic horizontal and moment loading, the analysis framework is established based on the same plasticity and CSSM framework used previously (Cocjin *et al.*, 2017). The long-term displacements of a mobile mudmat consist of (i) primary consolidation under the applied vertical load (which is essentially the self-weight of the structure and the foundation); (ii) plastic displacements due to the combined vertical-horizontal failure mechanism beneath the mudmat as it slides; and (iii) displacements associated with the dissipation of the excess pore pressure induced by the shear failure beneath the foundation, i.e., the shear induced consolidation. The displacements from the above three different mechanisms are combined to determine the final accumulated displacements.

Accumulated vertical settlement

The accumulated total vertical settlement δ_v can be calculated as

$$\delta_v = \delta_{cv} + \delta_{pv} + \delta_{sv} \quad (1)$$

where δ_{cv} is the primary consolidation settlement; δ_{pv} is the accumulated plastic vertical displacement after a certain number of cycles; δ_{sv} is the accumulated shear induced consolidation settlement after a certain number of cycles. Note all the above three components are time-dependent. A detailed explanation and the calculation method for each of the above three major components are provided in the following text.

Primary consolidation settlement

The time-dependent primary consolidation settlement can be conveniently expressed as Equation 2 (Feng and Gourvenec, 2016):

$$U = \frac{1}{1+(T/T_{50})^{-m}} = \frac{\delta_{cv}}{\delta_{cv,end}} \qquad 2$$

where U is the consolidation degree expressed in terms of the vertical settlement; $\delta_{cv,end}$ is the vertical settlement at the end of the primary consolidation which can be calculated from the one-dimensional consolidation theory that sums up the consolidation settlement of each layer with the added stress determined from the elastic solution ; T is the dimensionless consolidation time as expressed as $T = c_v t/B^2$ (with c_v and t being the consolidation coefficient and the actual consolidation time, respectively); T_{50} is the dimensionless consolidation time to achieve a 50% consolidation (i.e., U=0.5); and m is a fitting parameter. Appropriate values for T_{50}, m and c_v are given by Feng & Gourvenec (2016) and White *et al.* (2019).

The secondary consolidation effect is neglected in the current study due to: (1) for a typical mudmat design life of 20 to 30 years, the secondary consolidation may not have begun for deepwater GoM soft clays; (2) even the secondary consolidation begins, the magnitude of the secondary consolidation settlement is typically more than an order of magnitude smaller than the primary consolidation settlement for typical deepwater GoM clays.

Plastic displacements

In each cycle, a mobile mudmat overcomes the horizontal sliding resistance due to the available pipeline expansion or contraction force. Thus, plastic deformation is induced in the clay medium that leads to plastic displacements of the mudmat. These plastic displacements are commonly calculated from the failure envelope, F, of the mudmat by adopting an associated flow rule which is appropriate for undrained conditions. The failure envelope of a mudmat (Feng *et al.*, 2014) can be simplified to Equation 3 in the current study due to the in-plane loading with a constant vertical load V representing the self-weights of the mudmat and the structures:

$$F = (\frac{M_x}{M_{max}})^q \left[1 - a\frac{H_y}{H_{max}} + b(\frac{H_y}{H_{max}})^2\right] + (\frac{H_y}{H_{max}})^2 - 1 \quad (3)$$

where M_{max} and H_{max} are the maximum moment resistance and the maximum horizontal sliding resistance, respectively; q, a and b are parameters depending on the foundation geometry and the soil strength profile (Feng *et al.*, 2014). Note, the effect of the vertical load needs to be considered in determining M_{max} and H_{max} , i.e., M_{max} and H_{max} are functions of V. A detailed methodology to derive these parameters is given by Feng *et al.* (2014) based on numerical analysis for a range of soil conditions and mudmat shapes. Further analyses using the same methodology can be used to extend these solutions to other project-specific conditions.

Giving the horizontal sliding displacement of $h_{y,i}$ in the i^{th} cycle, the plastic vertical displacement $\delta_{pv,i}$ can be determined from the associated flow rule. This can be performed in a single increment because no work-hardening is introduced and V, H_y, M_x remain constant in one cycle. Therefore $\delta_{pv,i}$can be calculated as:$F(V, H_y, M_x) = 0$ 4 (a)

$$\delta_{pv,i} = \frac{\partial F/\partial V}{\partial F/\partial H_y} h_{y,i} \quad 4\ (b)$$

where the subscript "p" denotes the plastic displacements. Then the total accumulated plastic vertical settlement δ_{pv} is obtained by summing the plastic displacements in each cycle as below:

$$\delta_{pv} = \sum_{i=1}^{Cycles} \delta_{pv,i} \qquad 5$$

Note the plastic displacements usually differ from cycle to cycle, because of the evolution of the sliding resistance over the cycles, and the increase of the vertical bearing and moment resistance due to the consolidation effect (Feng and Gourvenec, 2015; Vulpe *et al.*, 2016). An implicit assumption made in Equation 5 is that the sliding distance is treated as plastic displacement and the elastic component is neglected. This simplified assumption is reasonable due to the small elastic displacement comparing to the plastic displacement when the mudmat "fails" horizontally.

Shear induced consolidation

For typical deepwater soft clays that are normally or lightly overconsolidated, positive excess pore water pressure will be generated once the soil is sheared, in particular the clays directly beneath a mobile mudmat are forced to fail, reaching a steady resistance that is referred to as the critical state in the terminology of CSSM. This excess pore water pressure (which is induced by shear failure instead of the vertical loads) will dissipate over time, and this not only induces additional settlement but also the clay will gain in strength. The design is complicated by the fact that the failure of the clay is limited to a thin layer (shear band) directly beneath the mudmat, while for clays in the deeper depth, although shear stress is mobilized and pore pressure generated, the critical state usually will not be reached for typical deepwater clays, where the strength of which usually increases linearly with depth. Another challenge comes from the determination of the soil stress state, e.g., the state relative to the critical state, in the subsequent cycles. These topics are addressed below.

Shear failure at first cycle

Following the CSSM framework in the direct simple shear case (Randolph *et al.*, 2012), as shown in Figure 2, the normal compression line (NCL) and the critical state line (CSL) are two parallel lines with the same slope of λ, while the unloading-reloading path is a line with a slope of κ in the $e - \ln{(\sigma'_v)}$ space (where e is the void ratio and σ'_v is the vertical effective stress). The NCL and CSL have intersections of N and Γ with the vertical axis at $\sigma'_v = 1kP_a$. For a normally consolidated (NC) clay without a previous shearing history, the position will be on the NCL with the current vertical effective stress σ'_{vo} as represented by point A, which is a simplification that ignores the effective stresses in other directions. Given the typical loading rate to the mobile mudmat and the low permeability of deepwater soft clays, the failure associated with each expansion and contraction event is in an undrained condition. Hence, once the mobile mudmat is forced to overcome the sliding resistance, the clay beneath the mudmat will follow a path horizontally from A, generating excess pore water pressure and reducing the vertical effective stress. Eventually the stress state migrates to the position B at a stress level of σ'_{cs} on the CSL, where failure is reached and no more changes in the excess pore pressure occur with further shearing. The distance between A and B represents the induced excess pore pressure, and the undrained shear strength S_u (Equation 6) can be determined from the CSSM procedure by employing the linear relationship between the shear stress and the vertical effective stress in the $\tau - \sigma'_v$ space at the critical state as shown in Figure 2 (where τ is the shear stress).

$$S_u = Mexp\left(\frac{\Gamma - N}{\lambda}\right)\sigma'_{vo} = Mexp\left(\frac{\Gamma - N}{\lambda}\right)(\gamma' z + I_\sigma \sigma'_p U) \quad 6$$

where M is the slope of the CSL in the $\sigma_v' - \tau$ space; z is the depth below the soil surface; γ' is the equivalent submerged unit weight of the soil; U is the degree of primary consolidation since the mudmat was placed on the seabed; σ_p' is the applied surface pressure from the mudmat and is assumed to be uniform across the mudmat surface; and I_σ is the vertical stress influencing factor determining the vertical distribution of the applied surface normal stress at the centerline beneath the mudmat. The common way to determining I_σ is based on an elastic solution, e.g., Das (2013) and Cocjin *et al.* (2017).

The generated maximum excess pore pressure (as represented by the distance from A to B) at the critical state $u_{max,z}$ is expressed in Equation 7 (recalling $\lambda = (\mathrm{N} - \Gamma)/\overline{AB}$ in Figure 2):

$$u_{max,z} = [1 - exp\left(\frac{\Gamma - N}{\lambda}\right)] \cdot (\gamma' z + I_\sigma \sigma_p' U) \qquad 7$$

At the surface (i.e., directly beneath the mudmat), the shear stress equals the undrained strength S_u at that elevation, but it decreases with depth; however, S_u usually increases with depth (as can also be seen from Equation 6). Therefore, the soil further below the mudmat will not fail and thus will not reach the critical state, and the generated excess pore pressure will be less than that predicted by Equation 7, and can instead be expressed generally as u_z. The theoretical solution for this excess pore pressure for a NC clay at the deeper depth is derived in Appendix A using a Modified Cam Clay (MCC) model (Roscoe and Burland, 1968). A simplified way for approximation is shown in Equation 8 following Cocjin *et al.* (2017):

$$\frac{u_z}{u_{max,z}} = (\frac{\tau}{S_u})^\beta \qquad 8$$

where β is a fitting parameter.

Combining Equation 6 and 8 gives:

$$\frac{u_z}{u_{max,z}} = [\frac{\tau_{surf} I_\tau}{Mexp(\frac{\Gamma - N}{\lambda})(\gamma' z + I_\sigma \sigma'_p U)}]^\beta \quad 9$$

where τ_{surf} is the shear stress at the surface and equals S_u at the surface; I_τ is the shear stress influencing factor determining the vertical distribution of the applied surface shear stress at the centerline beneath the mudmat and can be determined using the elastic solution from Holl (1941) and re-presented by Cocjin *et al.* (2017).

Combining Equation 6, 7, 9, and applying z=0 in Equation 6 (for the determination of Su at the surface) yields:

$$u_z = [1 - exp\left(\frac{\Gamma - N}{\lambda}\right)] \cdot [\frac{\sigma'_p U}{\gamma' z + I_\sigma \sigma'_p U} \cdot I_\tau]^\beta \cdot (\gamma' z + I_\sigma \sigma'_p U) \quad 10$$

Consolidation after first shear failure

Subjecting to shearing on the NCL, the soil migrates toward the CSL, and generates excess pore pressure u_z, which is shown in Equation 10 and is also represented by the distance $\overline{A'B'}$ in Figure 3. After complete dissipation of u_z, the change of the void ratio, Δe_z, can be calculate from one-dimensional consolidation theory and is expressed as:

$$\Delta e_z = -\kappa \ln\left(1 - \frac{u_z}{\sigma'_{vo}}\right) \quad 11$$

Equation 11 works no matter whether the soil has reached the critical state or not, as long as the shear induced excess pore pressure is fully dissipated. Equation 11 is consistent with previous studies of the settlement of normally consolidated clays following cyclic loading

(Feng and Gourvenec, 2016; Laham *et al.*, 2020; Ohara and Matsuda, 1988; Yasuhara and Andersen, 1991). However, Ohara and Matsuda (1988) found that the slope of the recompression line (i.e., κ in Equation 11) was in between the swelling line and virgin compression line from oedometer tests for Kaolinite clays. Yasuhara and Andersen (1991) also found that the volumetric strain due to the dissipation of the cyclically induced pore water pressure from direct simple shear (DSS) tests was larger than that calculated from Equation 11 if κ was determined from the recompression line from oedometer tests. For example, for Drammen clay, the selected value of recompression stiffness, κ, needs to be increased by 50% in order to match the DSS test results. The possible reason for this discrepancy is the cyclic loading not only generates an excess pore water pressure in the soil body, but also disturbs the clay structure (Yasuhara & Andersen 1991). Such an effect may not be directly relevant to the mudmat case because only a single cycle takes place prior to each recompression stage. However, it may be appropriate to increase κ and the calculated void ratio change in Equation 11 above that determined from an oedometer recompression stage.

Combining Equation 11 with 10 and recalling $\sigma'_{vo} = \gamma' z + I_\sigma \sigma'_p U$ yield the shear induced settlement as

$$\Delta e_z = -\kappa \ln \{1 - [1 - exp\left(\frac{\Gamma - N}{\lambda}\right)] \cdot [\frac{\sigma'_p U}{\gamma' z + I_\sigma \sigma'_p U} \cdot I_\tau]^\beta\} \qquad 12\ (a)$$

$$\delta_{sv,1} = \int_0^\infty \frac{\Delta e_z}{1 + e_{zo}} dz \qquad 12\ (b)$$

where $\delta_{sv,1}$ is the shear-induced vertical consolidation settlement (the subscript "1" denotes the first cycle); and e_{zo} is the initial void ratio at depth z.

Shear induced consolidation in subsequent cycles

The behavior of soils in the subsequent cycles becomes much more complicated due to the following facts: (1) the undrained shear strength will generally increase depending on the depth and the generated excess pore pressure in the previous cycles; (2) Equation 8 may not work or the exponent β needs to be adjusted, because following the consolidation, the clay status departs from the NCL and migrates toward the CSL. Thus, the initial response will be elastic with no excess pore pressure, while Equation 8 is established based on the NC clays; and (3) the shear stress influence factor I_τ from the conventional elastic solution may not necessary valid (this is also true for the first cycle) and it is difficult to estimate the accuracy of the elastic solutions. For the practical purpose, the above factors are simplistically and implicitly addressed as follows.

Referring to Figure 3, at the surface where the soil has reached the critical state at the first cycle, the change of the void ratio in the subsequent consolidation (from B to C) is linearly proportional to the distance $\overline{AB}$. Similarly, the change of the void ratio in the consolidation after the second shearing is linear proportional to the distance $\overline{CD}$. It is straightforward to obtain the relation between $\overline{AB}$ and $\overline{CD}$ as follows (if the vertical effective stress remains the same):

$$\frac{\overline{CD}}{\overline{AB}} = 1 - \frac{\kappa}{\lambda} \qquad 13$$

Therefore, it can be concluded that at the surface, the change of the void ratio in the next shear induced consolidation is a portion of that occurred in the previous shear induced consolidation with a factor of $(\lambda - \kappa)/\lambda$. In deeper depth, the behavior is complicated by the difficulty in determining of the excess pore pressure as described before; however, it is simplistically assumed that the change of the void ratio in subsequently shear induced consolidation at deeper depths follows the same rule as in the surface (see the path $A' - B' - C'$) as shown in Figure 3. The above assumption, together with the assumption that the vertical effective stress remains the same, leads to the conclusion that the shear induced vertical consolidation settlement due to the subsequent shear cycles and consolidations decays geometrically with a factor of $(\lambda - \kappa)/\lambda$, as shown below:

$$\delta_{sv} = \sum_{i=1}^{Cycles} \delta_{sv,1} \left(1 - \frac{\kappa}{\lambda}\right)^{i-1} = \delta_{sv,1} \left[\frac{\lambda}{\kappa} - \frac{\lambda}{\kappa}\left(1 - \frac{\kappa}{\lambda}\right)^{Cycles}\right] \quad 14$$

where δ_{sv} is the total vertical settlement from the shear induced consolidation. Note Equation 14 implicitly assumes that all the shear induced excess pore pressure has been dissipated before next cycle. This assumption is reasonable from the practical operation of view for three reasons: (1) the full consolidation assumption leads to a higher shear induced consolidation settlement and thus is in general conservative for the current application; (2) usually the pipeline will be in production for months to years before the next full shut-in/start-up cycle and thus it is expected that quite a substantial portion of the shear induced pore pressure will dissipate; (3) the operative consolidation coefficient c_v for surface foundations under lateral shearing is usually significantly higher than the one measured from oedometric tests, especially under cyclic loading due to the change of the soil stiffness and stress influence zone etc. White *et al.* (2019) comprehensively reviewed the model tests on the operative c_v and found the operative c_v under horizontal loading can increase by a factor of 10 under cyclic loading, and the operative c_v for pipelines under axial shearing is on average eight times greater than the one from oedometric tests. Similar findings have also been reported by Krost *et al.* (2011).

Accumulated rotation

The estimation of the long-term rotation follows the same framework as for the estimation of the long-term vertical settlement, and is expressed as below:

$$\theta_x = \theta_{cx} + \theta_{px} + \theta_{sx} \tag{15}$$

where θ_x is the long-term total rotation about the x-axis; θ_{cx} is the rotation due to the primary consolidation; θ_{px} is the accumulated plastic rotation after a certain number of cycles; θ_{sx} is the accumulated rotation due to the shear induced consolidation after a certain number of cycles. The same with the vertical settlement, all the above three rotation components are time-dependent.

The plastic rotation can be estimated from an associated flow rule similar to Equation 4 as shown below:

$$\theta_{px} = \sum_{i=1}^{Cycles} \left(\frac{\partial F/\partial M_y}{\partial F/\partial H_y} h_{y,i} \right) \tag{16}$$

The rotation from the shear induced consolidation is simplistically estimated from the expected difference in the vertical settlements of two mudmats, which have half of the length of the original mudmat under certain vertical loads (see Figure 4). As shown, the original mudmat is divided into two mudmats with half of the original length by the center line with vertical loads V_L (acting 0.25L to the right of the left edge of the mudmat) and V_R (acting 0.25L to the left of the right edge of the mudmat), which are separated by a distance of 0.5L. V^L and V^R are determined as follows:

$$V^L = \frac{V}{2} - \frac{M_x}{0.5L} \tag{17 (a)}$$

$$V^R = \frac{V}{2} + \frac{M_x}{0.5L} \quad 17\text{ (b)}$$

where M_x is the overturning moment along the x-axis caused by the horizontal load excluding the overturning moment from the eccentricity of the vertical load, because the eccentricity from the vertical load will be taken into account in the rotation from the primary consolidation. Therefore, the vertical load on the right half-length mudmat exceeds the left one by $4M_x/L$, and the differential settlement is the settlement of the right half-length mudmat under the vertical load of $4M_x/L$, and the rotation from the shear induced consolidation can be estimated as follows:

$$\theta_{sx} = \frac{\delta_{sv}^R}{0.5L} \quad 18$$

where δ_{sv}^R is the vertical settlements from the shear induced consolidation of the right half mudmat under vertical load of $4M_x/L$ (it is hypothetically imagined that these left and right half mudmats are not connected). The calculation of δ_{sv}^R follows the same procedure described in the section "Shear induced consolidation" except the vertical load is replaced by $4M_x/L$. This conceptual model for the shear induced rotation estimate is a simplification but is judged to be conservative for design because the two hypothetical half-length mudmats are actually rigidly connected. This neglected structural connection will reduce the shear-induced rotation, and so the approach set out here in conservative.

The rotation from the primary consolidation can follow the exact same method as for the rotation from the shear induced consolidation, simply by determining the M_x from the eccentricity of the vertical load. For subsea mudmats at deepwater sites with soft clays, the mudmat stiffness is close to be rigid compared to the soil stiffness. Thus, the rotation (and the differential settlement) from the primary consolidation is usually small and is typically neglected in current design practice due to the controlled structural self-weight eccentricity. A rule of thumb for this rotation based on the available operational observations and writers' experience is that the differential settlement from the primary consolidation is less than 20% of the total primary consolidation at the center, which can be used to estimate the rotation. In this study, this rule of thumb value is used for estimating the rotation from the primary consolidation for the sake of illustrating the design process; however, detailed scrutiny and analysis may be required for a particular project under complex soil conditions or the mudmat geometry and eccentricity are out the range of normal practice.

Analysis Framework Input Parameters

Table 1 lists the input parameters required for the proposed framework. The common challenge of using the critical state type soil models in practice is the difficulty in determining the key critical state parameters, i.e., $N, \Gamma, \lambda, \kappa, M$. These parameters are not routinely measured in standard laboratory testing programs in an offshore project delivery, especially in the United States. Thus, it is valuable to establish generic values for these critical state parameters for GoM deepwater sites. Table 1 provides the typical values/best estimate for deepwater soft clays in the GoM, and Figure 5 show the laboratory test data from five different deepwater locations in the GoM. Note in Figure 5, the high estimate and low estimate values are also provided based on both the standard statistical analysis and the engineering judgement. The soil samples used in the laboratory were retrieved using a thin-walled Shelby tube with a push sampler, minimizing the disturbance based on the state-of-the-practice. One-dimensional consolidation tests and Ko-consolidated direct simple shear (DSS) tests were usually performed. In Ko-DSS tests, the undisturbed soil samples were usually consolidated to a normal stress of about 1.5 to 2 times the in-situ vertical effective stress to ensure the soil is normally consolidated before shearing. The determination of the input parameters from available laboratory tests is explained as follows.

N and λ were determined from the relationship between the void ratio e and the logarithmic vertical effective stress σ_v' from NC clays assuming a linear relationship between the two exists as shown in Figure 5 (a). M is the ratio of the undrained shear strength to the vertical effective stress at failure (see Figure 2) and was determined from the Ko-DSS tests on NC clays as shown in Figure 5 (b). κ is back calculated by assuming $\lambda/\kappa = C_c/C_r$, where C_c/C_r was determined from both engineering judgement and oedometer tests (C_c is the slope of the virgin compression line from oedometer tests, while C_r is the slope of the unloading-reloading line) as shown in Figure 5 (c) with the best estimate of about 6.5. Γ is not measured from the laboratory tests but is calculated from CSSM conditioned on the measured value of S_u/σ_{vo}' following Equation 6 as follows:

$$\Gamma = \mathrm{N} + \lambda \ln\left[\frac{(S_u/\sigma_{vo}')_{NC}}{M}\right] \tag{19}$$

where $(S_u/\sigma_{vo}')_{NC}$ is the normalized stress ratio for NC clays from Ko-DSS tests and is determined to be around 0.27 at shallow depths in deepwater GoM as shown in Figure 5 (d).

Other parameters in addition to the critical state parameters are determined as follows. The submerged unit weight is usually within about 3 kN/m^3 to 5 kN/m^3 in the top 10 m. The consolidation coefficient c_v value is correlated to the water content following the design chart (NAVFAC, 1986), and usually consistent with the laboratory tests on soil samples, e.g., the consolidation tests from the low pressure direct shear tests (Geoconsulting, 2015). T_{50} and m (see Equation 2) for the primary consolidation are based on Feng and Gourvenec (2015) and White *et al.* (2019); similar results were also obtained from in-house simulations. In some soils, there can be a strong variation in permeability and stiffness with stress level and loading path, causing c_v to vary with depth and through time; these effects are reviewed by White et al. (2019). In soils where this is a significant influence, the adopted value of c_v could be calibrated from a consolidation analysis with a more sophisticated model for stiffness and permeability in which these additional effects are considered. β (see Equation 8) is fitted to the excess pore pressure path for the NC clay obeying the MCC model (see Appendix A) for typical values of λ/κ.

Analysis Example

A practical analysis example is provided in this study as shown in Figure 6. The example mudmat has a length of 7.62m (25 feet) and a width of 6.1m (20 feet) with no skirt and the geometric center "O" is also shown in the figure. The total vertical load (including the self-weight of the mudmat) is about 400 kN with an eccentricity e_y=0.61m (2 feet). The moment arm of the horizontal load/displacement is h_0=0.91m (3 feet). The mudmat is forced to move by 51mm (2 inches) in the y direction (along the length direction of the mudmat) during shut-in when the pipeline contracts (the total contraction of the pipeline during shut-in is typically greater than 51mm, but the majority of the contraction will be absorbed by the slider. Hence, only the portion of the contraction that has not been absorbed by the slider will force the mudmat to move). During start-up, it is assumed the pipeline is in a state of free expansion because the expansion will be totally absorbed by the slider and the frictional force in the slider is minor. Therefore, the horizontal sliding distance of the mudmat itself is 51mm (2 inches) in each cycle happened during shut-in when the pipeline contracts. It is planned that the pipeline will have six full cycles of shut-in and start-up (i.e., complete cool down of the pipeline to the ambient temperature after shut-in, and the heat up to the design temperature of the fluid after start-up) each year. Partial cycles of shut-in and start-up have been neglected because the expansion and contraction will be absorbed by the slider and hence will not induce horizontal sliding distance.

The best estimate of the in-situ soil undrained shear strength is 2.15 kP_a (45 psf) at the mudline and increases with depth with a gradient of 0.94kP_a/m (6.0 psf/ft). The submerged unit weight of the soil is assumed to be constant of 3.9 kN/m^3 (25 pcf). $N, \Gamma, \lambda, \kappa, T_{50}, m$ are taken to be the best estimate GoM values (see Table 1). M is taken to be 0.45, β is assumed to be 2.75 based on the typical values of λ/κ. c_v is taken to be 0.68 m^2/year.

The long-term primary consolidation settlement $\delta_{cv,end}$ (where the subscript "end" denotes the end of the primary consolidation) is calculated following the conventional oedometer design procedure, and is estimated to be 584 mm with the void ratio simplistically calculated from the vertical effective stress from the $e - \ln(\sigma'_v)$ space. The differential settlement of the mudmat is assumed to be 20% of total primary consolidation settlement giving 117 mm. This differential settlement gives a long-term rotational angle of $\theta_{cx,end}$ about the x-direction of 0.9°. These long-term primary consolidation settlement $\delta_{cv,end}$ and rotation $\theta_{cx,end}$ can be used to estimate the consolidation settlement δ_{cv} and rotation θ_{cx} at any time after installation with the aid of the time-dependent consolidation degree as shown in Equation 2.

The plastic displacements are straightforward to calculate following an associated flow rule (see Equation 4 and 16); however, the challenge is that the soil strength and hence the undrained capacity of the mudmat under six-degree of freedom changes with time, e.g., the uniaxial vertical bearing capacity and overturning moment can increase by 50%, and the uniaxial sliding resistance can be doubled after the full consolidation depending on vertical loading (Feng and Gourvenec, 2015). Therefore, a rigorous approach may be pursued by considering the time each sliding failure occurred with the accomplished consolidation degree at that specific time; however, sensitivity studies show that the induced plastic displacement decreases with the increase of the mudmat consolidation degree under vertical loading (and hence the increase of the mudmat undrained capacity), and it is difficult to capture the exact operational timing that the full shut-in and start-up cycle will happen, because the actual pipeline operation is affected by various uncertain factors that is usually very difficult to predict during the design phase. Thus, for the design purpose, it is assumed that the plastic displacements remain the same for all the cycles and can be estimated from the first cycle with no increase of the mudmat undrained capacity with time. This assumption significantly simplifies the calculation by eliminating the determination of the consolidation degree at the specific time that the pipeline will shut-in or start-up, which is highly uncertain in practice. In addition, this assumption is on the conservative side for typical designs because for a mobile mudmat, the critical design criterion is the displacement tolerance instead of the capacity of the mudmat (Note, it is assumed here that the anchor forging which is the load transfer mechanism between the pipeline and the mudmat has sufficient capacity to effectively transfer the load from the pipeline to the mudmat. Thus, proper design checks need to be performed to ensure the anchor forging has the required capacity). Based on the above discussion, given a 51mm (2 inches) sliding distance per cycle, the vertical plastic displacement is estimated to be $\delta_{pv} = 1.2\ mm\ (0.048\ inches)$ per cycle, and the plastic rotation about the x-axis is $\theta_{px} = 0.013°$ per cycle.

For the shear induced consolidation, sensitivity studies show that the induced displacements are largest when the primary consolidation is completed. Given the surface pressure of 8.6 kP_a (180 psf) (it is assumed that the vertical load of 400 kN is uniformly distributed over the mudmat surface) with 100% primary consolidation, the shear induced consolidation settlement after first cycle is $\delta_{sv} = 5.3\ mm\ (0.21\ inches)$ following Equation 12, and decreases geometrically by a factor of $(\lambda - \kappa)/\lambda = 0.827$ with cycles (assuming in each cycle, the shear induced pore pressure has dissipated completely). After the full consolidation under the given vertical loading, the undrained sliding resistance of the mudmat following Equation 6 is about 2.3 kP_a (48 psf). This undrained shear strength gives the sliding load at failure of 107 kN, the overturning moment at failure $M_x = 107 \times 0.91 = 97\ kN \cdot m$ and $4M_x/L \approx 51\ kN$, which will be used in Equation 18 to estimate the rotation due to the dissipation of the shear induced pore pressure. This $4M_x/L \approx 51\ kN$ gives δ_{sv}^{R}=1.7 mm (0.075 inches) per cycle (i.e., the vertical settlement from the shear induced consolidation of the right half mudmat under the load $4M_x/L$ and increased by 50% to reflect the discrepancy between the calculation and the DSS tests as discussed before), and hence gives the rotation from the shear induced consolidation $\theta_{sx} = 0.03°$ per cycle. Similar to the shear induced consolidation settlement, the shear induced consolidation rotation θ_{sx} decreases geometrically by a factor of $(\lambda - \kappa)/\lambda = 0.827$ with cycles based on the current assumptions.

Figure 7 and 8 show respectively the vertical settlement and rotation with time after installation for the design case (i.e., the mudmat slides 51mm per cycle and six cycles per year). For simplicity, it is assumed the pipeline will be in operation as soon as the mudmat is installed (In practice, the first start-up of the pipeline will be typically 3 to 12 months after the mudmat is in place). As shown, for the accumulated vertical settlement, the majority of the vertical settlement comes from the primary consolidation, which contributes to about 75% of the total settlement for a design life of 20 years. The vertical displacement due to the plastic failure contributes to the total settlement by about 20%, while the shear induced consolidation settlement contributes to the rest 5% of the total settlement. For the accumulated rotation, the rotation from the plastic failure becomes the dominant factor that contributes to more than 60% of the total rotation, while the rotation from the differential settlement in the primary consolidation contributes to about 30%, and the remaining less than 10% comes from the shear induced consolidation.

The key finding from this design example is that the displacements from the shear induced consolidation are likely an order of magnitude smaller than the anticipated total displacement, which makes the sophisticated modeling of the complex mechanism associated with the shear induced consolidation for a partial mobile mudmat likely unnecessary. However, if a fully mobile mudmat without a sliding mechanism that absorbs the majority of the expansion/contraction, the modeling of the shear induced consolidation may become important (Cocjin *et al.*, 2014; Cocjin *et al.*, 2017). Another learning from this design example is that knowing the primary consolidation settlement and rotation, it is likely the total displacement can be upper bounded by multiplying a factor of, on the order of, 1.5 to the primary consolidation settlement and rotation for a partially mobile mudmat.

This finding can be quite useful for engineering screening purpose during the concept selection, although the finding in this study is still preliminary and sensitivities need to be performed in the detailed design phase. In comparison, the preliminary design experience from fully mobile mudmats indicates that the long-term accumulated settlement from cyclic loading could be about 5 to 10 times the primary consolidation settlement.

Discussions and Recommendations

Compared to the existing studies on fully mobile subsea mudmat, the current study possesses a number of features: (i) the current study focuses on a partially mobile subsea mudmat that takes the advantage of the sliding mechanism in the PLET which absorbs the majority of the pipeline expansions/contractions. Therefore, the sliding movement of a partially mobile subsea mudmat in one cycle (on the order of few inches) is significantly lower than a fully mobile mudmat (on the order of tens of feet), and represents a fit-for-purpose engineering solution with significant commercial competitiveness between a fully anchored mudmat (which may be too large causing installation issues or need corner piles to anchor) and a fully mobile mudmat (which may cause excessive settlements that compromise the structural integrity in deepwater soft clayey deposits); (ii) a predictive method for mudmat rotations, which has not yet been fully explored in the existing studies, is proposed with the intention to close out the last design concern for a partially mobile mudmat; (iii) typical ranges of soil properties pertinent to using the proposed framework are presented for deepwater GoM soft clays to facilitate the practical design, and in particular to provide a basis for sensitivity study for creating the risk matrix of a partially mobile mudmat.

However, the above framework is not intended to model the complex behavior of the soil under a partially mobile mudmat precisely. Instead, the key motivation is to provide a framework that can be used to capture the key mechanism of a partially mobile mudmat and to retain the simplicity and rigor of the model for engineering concept selection. The idea behind the simplification in modeling is that the uncertainties in the input soil parameters and the simplified assumptions can be conveniently captured from sensitivity studies, and a range of predictions from low estimate to high estimate can be constructed with reasonable engineering efforts to build up the risk matrix for decisions. Nevertheless, the simplifications do not necessarily rule out the opportunity to improve the modeling of the framework. The following list describes the recommendations for future improvement of the current framework (with the high priority one comes first based on the writers' judgement):

1. Further physical model test programs, e.g., centrifuge tests, 1-g reduced scale model tests, will be beneficial to further improve the understanding of the mechanism of a partially mobile mudmat and to further refine the current predictive framework (the writes are not aware of any model tests existed in the public domain on partially mobile mudmats up to date). In addition, it is highly recommended that the operators start to collect the field monitoring data from existing deployments that the current framework can be benchmarked to.
2. The current framework for the mudmat rotation based on the one-dimensional consolidation is simplified and incompatible with a rigid foundation where the two half-length mudmats (see Figure 4) are calculated to have different settlements and used in the rotation prediction. It is judged that the current method is on the

conservative side because the two "hypothetical" half-length mudmats are actually rigidly connected that can help reduce the differential displacements. Therefore, a more rigorous predictive framework can be highly beneficial to reduce the conservatism in the current model.

3. The soil strength S_u used in predicting the displacements associated with the plastic failure is based on the typical values measured from in-situ tests, which usually has a finite value at the mudline. However, the S_u used in predicting the shear induced consolidation displacements is directly calculated from the theoretical model based on the critical state concept (i.e., Equation 6) under the given vertical loading with a certain consolidation degree, which always gives a zero strength at the mudline if there is no vertical loading or the consolidation has not yet occurred under the given vertical loads. Apparently, with the intention of simplification and conservatism, there is an inconsistency in the current framework in selecting the S_u in predicting the plastic failure and shear induced consolidation. In addition, a constant submerged unit weight for soil assumed in the current study is in general inconsistent with the void ratio predicted from CSSM and may need to be improved. Therefore, a more rigorous approach is probably required, e.g., to trace the variation of the in-situ S_u after the first failure and with time and to use a consistent S_u and submerged unit weight profiles in the all the predictions.

4. An associated flow rule is used in the current framework without further validations. Existing studies show that a non-associated flow rule is more appropriate for mudmats and can result in higher vertical settlement from the associated flow rule (Cocjin *et al.*, 2017). However, the sliding displacement of a fully mobile mudmat in each cycle (on

the order of tens of feet) is significantly larger than the one in a partially mobile mudmat proposed here (on order of few inches). Therefore, further research may be granted to study the appropriateness of the associated flow rule for predicting the displacement associated with the plastic failure.

5. The CSL in the current framework is a fixed line in the $e - \ln(\sigma'_v)$ space following the conventional CSSM; however, the CSL for actual clays might migrate under shearing (e.g., remolding) or be curved in the low stress regions (Cocjin *et al.*, 2017). This simplification can be further improved by introducing a curved CSL and/or let the CSL migrate with loading. Alternatively, the effect of a migrating CSL can be assessed with the current framework that the intercept of the CSL with the vertical axis in the $e - \ln(\sigma'_v)$ space, Γ, can be adjusted based on different values of $(S_u/\sigma'_{vo})_{NC}$ covering both intact and remolded conditions.
6. Linear superposition of the settlement/rotation from three different mechanisms is used in the current framework. The main intention is to keep the framework simple, but the key difficulty is to find the interactions between the various mechanisms. It is unclear to the authors how these interactions can be efficiently captured, but the authors judge the interaction is likely not strong. The reason is that the primary consolidation settlement is contributed a lot by the soil in the deeper depth, while the plastic sliding failure only involves a thin layer directly beneath the mudmat. Thus, the current judgment is that the linear superposition is appropriate and is likely on the conservative side.

Conclusions

This study provides a displacement predictive framework for practical designs of partially mobile subsea foundations (i.e., mudmats), which represents a fit-for-purpose engineering solution with significant commercial competitiveness between a fully anchored mudmat (which may be too large causing installation issues or need corner piles to fix the foundation) and a fully mobile mudmat (which may cause excessive settlements that compromise the structural integrity) in deepwater soft clayey sites. The partially mobile mudmat takes the advantage of the PLET's sliding mechanism that absorbs the majority of the pipeline expansions and contractions, and therefore minimizes the cyclic shearing to soils from the sliding mudmat.

The proposed predictive framework divides the accumulated displacement field beneath a sliding mudmat into three components: (i) primary consolidation induced displacement; (ii) plastic displacement when the sliding mudmat exceeds the soil strength in each cycle; and (iii) shear induced consolidation displacement, i.e., the displacement associated with the dissipation of the shear induced excess pore water pressure. The primary consolidation is predicted following the conventional one-dimensional consolidation theory, while the plastic displacement is predicted from the failure envelope of a mudmat subjected to six-degree of freedom (Feng *et al.*, 2014) using an associated flow rule, and finally the shear induced consolidation is predicted based on the critical state soil mechanics. Simplified analytical solutions for the accumulated vertical settlements and rotations under multiple cycles are derived, and the typical ranges for the key input parameters are provided for the GoM deepwater soft clays to facilitate practical designs. A typical design example is provided, and preliminary finding is that the dominant accumulated displacements come from the primary consolidation and plastic failure, while the shear induced consolidation is about an order of magnitude less than the total accumulated settlement in a typical design life.

The proposed framework is not intended to model the complex behavior of the soil beneath a partially mobile mudmat precisely. Instead, the key motivation is to provide a framework that can be used to capture the key mechanism of a partially mobile mudmat and to retain the simplicity and rigor of the model for engineering concept selection. Opportunities for further refinement of the framework are listed, in particular, the role of physical model tests and field monitoring are emphasized to better improve the predictive framework and to quantify the uncertainties of the predictive framework.

Data Availability Statement

The following data, models, or code generated or used during the study are available from the corresponding author by request: Derivation of the associated flow rule from the mudmat failure envelope.

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Appendix A Derivation of Excess Pore Water Pressure Based on The Modified Cam Clay Model

The modified Cam Clay (MCC) model is formulated in the $e - ln(p')$ space, where p' is the mean effective stress. By using the undrained condition, following equation can be obtained with zero volumetric strain:

$$\kappa' ln\left(\frac{p'}{p'_0}\right) = (\kappa' - \lambda') ln\left(\frac{p'_c}{p'_0}\right) \quad \text{A1}$$

where p'_0, p'_c are the initial mean effective stress and the preconsolidation pressure, respectively. κ' and λ' are the slope of the recompression and normal compression lines in the $e - ln(p')$ space.

Manipulating Equation A1 gives:

$$p'_c = p'_0 \left(\frac{p'}{p'_0}\right)^{\kappa'/\kappa' - \lambda'} \quad \text{A2}$$

With the relationship of $p'_{cs} = 0.5 p'_c$ (where p'_{cs} is the critical state pressure) for the MCC model, the undrained shear strength Su is determined as:

$$S_u = 0.5 M' \left(\frac{1}{2}\right)^{1 - \frac{\kappa'}{\lambda'}} p'_0 \quad \text{A3}$$

where M' is the slope of the critical state line (CSL) in $q - p'$ space where q is the deviatoric stress, and can be determined from the yield surface of the MCC model as:

$$q = M' \sqrt{p'(p'_c - p')} \quad \text{A4}$$

Combining Equation A2, A3, and A4 gives

$$\frac{\tau}{S_u} = 2^{1-\frac{\kappa'}{\lambda'}} \cdot \frac{p'}{p'_0} \sqrt{\left(\frac{p'}{p'_0}\right)^{\lambda'/\kappa'-\lambda'} - 1} \qquad \text{A5}$$

where τ is the shear stress defined as $0.5q$.

The excess pore water pressure u is defined as the difference between the initial and current mean effective stress as $u = p'_0 - p'$, and is related to the maximum excess pore water pressure u_{max} as

$$\frac{u}{u_{max}} = \frac{p'_0 - p'}{p'_0 - p'_{cs}} \qquad \text{A6}$$

Combining Equation A2 and A6 and using $p'_{cs} = 0.5p'_c$ give

$$\frac{u}{u_{max}} = \frac{1}{1-2^{\frac{\kappa'}{\lambda'}-1}}\left(1 - \frac{p'}{p'_0}\right) \qquad \text{A7}$$

Therefore, the relationship between $\frac{\tau}{S_u}$ and $\frac{u}{u_{max}}$ following the MCC model is given by Equation A5 and A7, and is plotted in Figure A1 together with Equation 8 (note the slopes of the normal compression and recompression lines in the $e - ln(p')$ space will be different from those in the $e - \ln(\sigma'_v)$ space, but the ratio of the slopes of the two lines remain the same in the two spaces).

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List of Tables

Table 1 Framework input parameters

	Parameter	Illustration	Typical values for deepwater GoM
Geometry	B	mudmat width	3m - 8m
	L	mudmat length	7m – 15m
Loading	V	vertical load on mudmat	Varies
	e_y	vertical load eccentricity	Varies
	h_0	moment arm of horizontal load	Varies
	h_y	horizontal sliding distance in each cycle	less than 150mm
Soil model	γ'	submerged unit weight of soil	3 kN/m^3 to 5 kN/m^3
	N	Intersection of NCL with vertical axis at unit normal stress (in kPa)	4.6 (best estimate)
	Γ	Intersection of CSL with vertical axis at unit normal stress (in kPa)	4.3 (best estimate)
	λ	slope of NCL in $e - ln(\sigma'_v)$ space	0.58 (best estimate)
	κ	slope of recompression line in $e - ln(\sigma_v')$ space	0.1 (best estimate)
	M	Slope of CSL in $\tau - \sigma_v'$ space	0.3-0.5
	β	Excess pore pressure fitting parameter	2.5-3.0
	c_v	consolidation coefficient (for primary consolidation only)	0.4-2.0 m^2/year
	T_{50}	normalized consolidation time to achieve 50% pf consolidation	0.043
	m	Consolidation degree fitting parameter	1.05

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