Whole Life Design: Theory and Applications of This New Approach to Offshore Geotechnics

Susan Gourvenec

Received: 29 November 2021 / Accepted: 31 May 2022 / Published online: 5 July 2022
© The Author(s) 2022

Abstract Geotechnical properties can evolve throughout the design life of a structure due to actions imposed during installation, the operational life or late and end of life management of an asset. Whole life geotechnical design seeks to predict soil-structure responses across the design life by considering the whole life of imposed actions coupled with geotechnical properties that evolve with each action. In contrast, traditional geotechnical design considers the ‘worst case’ single value of minimum resistance or stiffness coupled with the ‘worst case’ single value of maximum action over the design life. The emerging philosophy of whole life geotechnical design checks limit states at different stages of the ‘whole life’ against ‘current’ geotechnical properties, updated based on the processes that have occurred and the responses that have accumulated earlier in the design life. Consideration of whole life geotechnical response provides greater insight, enabling forecasting of the response of a supported structure through and beyond its design life. Insights can be applied at the initial design stage for optimal sizing; through life for assessing or predicting cumulative displacements or changes in resistance, and assumptions in the initial design against observed performance. By extension, these insights can be used to predict actual remaining design life; for relifing or re-purposing; and decommissioning. This paper presents the overarching philosophy of whole life geotechnical design, theory underpinning the evolution of geotechnical properties, derivation of the appropriate parameters, and some applications. This paper demonstrates the potential of whole life design, particularly to the emerging opportunities of offshore renewable energy infrastructure.

Keywords Geotechnical response · Geotechnical design · Whole life

Introduction

Philosophy of Whole Life Geotechnical Design

The philosophical basis for whole life geotechnical design is that design outcomes can be improved by considering whole life actions and the resulting whole life soil responses [1]. Soil responses can be influenced by changes in the soil surface profile and/or changes in soil properties from imposed actions during the life of a structure, which affect the response of the soil and the structure to subsequent actions. As such, whole life design checks limit states by comparing ‘current’ imposed actions with ‘current’ geotechnical properties, i.e. which have been updated based on preceding actions and responses.

The fundamental basis of whole life design is the same as that for traditional geotechnical design—i.e. that a design action does not cause a design limit state to be exceeded. However, in a whole life design approach, this check is made repeatedly, throughout the life of the structure or system (Eq. 1), rather than just once for the worst case combination of maximum action and minimum resistance or stiffness, as is generally the case for traditional offshore geotechnical design (Eq. 2).

\[
\text{WLD} : \text{Design action } (F, d)_t \leq \text{Design limit state } (F_{\text{lim}}, d_{\text{lim}})_t
\]
Conventional: Design action \((F, \delta)_{\text{max}}\) \(\leq\) Design limit state \((F_{\text{lim}}, \delta_{\text{lim}})_{\text{min}}\) (2)

**Figure 1** illustrates schematically the varying imposed actions and geotechnical response across the design life of a structure that underpin the philosophy of whole life geotechnical design. The imposed actions cover a temporal spectrum. They may be short-term ‘events’ (e.g. undrained installation, retrieval or an impact or snag load); longer ‘episodes’ within the broader context of the whole life experiences that may be comprised of a series of events (e.g. construction or operational processes and extreme weather events); or over the whole life of a structure (e.g. periodic thermal events (freeze–thaw, seasonal cycles, operational heating and cooling) or ratcheting. Events or episodes can be superimposed on a background of the whole life actions and response to enable greater scrutiny of specific activities or environmental influences.

It is essential to understand current operative soil strength as well as the current position of a structure to inform predictions of the geotechnical response for an event during the life of the structure. Questions such as “what is the soil strength and stiffness at the start and end of the episode ‘B’ in Fig. 1?” determine the (true) geotechnical stability of the structure during the episode and subsequently for future events, such as ‘C’ and ‘D’. From Fig. 1, it is clear that the design resistance for WLD (shown in bold solid line [green]) always exceeds the current design action (shown in fine solid line [grey]), while adopting the maximum action and minimum resistance across the design life leads to the design resistance for the traditional approach to fall below the design action, i.e. violating the failure criterion that the design action does not exceed a design limit state (final dashed line [red]). In practice, this would result in a larger foundation or anchor being required, increasing the challenges associated with installation and cost, both financial and embodied carbon. At the decommissioning stage, should the structure be removed from the seafloor, the whole life design approach enables a realistic prediction of resistance in order to determine required crane or vessel capacity, while a traditional design approach would considerably under predict the actual uplift capacity. Identifying the ‘true’, or current, operative shear strength and stiffness to inform that calculation enables a more realistic prediction of geotechnical resistance and optimized design, compared to assuming that the initial (in situ) or cyclically degraded properties apply throughout the life. In turn, this approach provides more accurate predictions of the system reliability (which can be quantified if uncertainty levels are applied to the actions and resistances) and therefore a better analysis of geotechnical risk.

The geotechnical concepts underpinning whole life geotechnical design are not new, and aspects of whole life design are established in onshore and offshore geotechnical practice, although perhaps without being named as such. Staged embankment construction (e.g. [2]) and tailings dam design (e.g. [3]) onshore are based on whole life design principles, with consolidation as a driver to improve
resistance for a further stage of construction. This has been referred to as ‘ongoing design’ in the context of tailings dams [3]. In offshore geotechnical practice, in its simplest form, whole life geotechnical design is the reliance on self weight consolidation beneath a structure between installation and the winter storm season (e.g. [4]), or provision of scour mats around offshore foundations having identified mobility of the seabed during the operating life [5]. However, whole life geotechnical design is emerging as a more widely applied philosophy, to encompass approaches that improve design outcomes by predicting evolving soil parameters due to imposed actions during the design life.

This keynote paper seeks to highlight and define the concept of whole life geotechnical design as it applies in offshore engineering and to demonstrate the versatility for applying whole life design concepts to different types of design analyses, seabed types and geotechnical structures, for both simple and complex loading conditions.

**Versatility of Whole Life Geotechnical Design Approach**

Whole life geotechnical design is not a single tool for a single application, but an approach that can be applied to a variety of different types of design analyses at different stages of the design life and beyond.

**Application to Different Design Types**

Whole life geotechnical design can be applied to ultimate limit state, serviceability limit state and fatigue limit state design analyses:

- For ultimate limit state (ULS) design, i.e. stability, soil response may cause the available resistance to vary due to earlier actions.
- For serviceability limit state (SLS) design (e.g. settlement or rotations), soil response may cause the stiffness under an action to vary due to earlier actions.
- For fatigue limit state design (e.g. of piles), the soil response may cause the stiffness and the cyclic structural stresses, to evolve over the design life, due to earlier actions, thus improving the fatigue life by varying the fatigue ‘hot spot’ through life.

The soil response causing changes in resistance and stiffness over time may be consolidation due to changes in imposed action, either monotonic or cyclic; sediment transport (scour and erosion) leading to a change in seafloor profile; or cyclic strain—e.g. ratcheting, or accumulation of deformation, under cyclic actions (Fig. 2). In fine grained soils, the rate at which geotechnical properties evolve will also vary through life with changes in coefficient of consolidation, which is linked to stiffness and permeability.

**Application to Different Stages of Design Life**

Whole life geotechnical design can yield value across and beyond the design life of a structure from the initial design stage (e.g. for sizing foundations and anchors) through to informing decommissioning options, e.g. the required crane capacity to assess the feasibility of removal, or the stability of the structure if left in situ beyond the design life [6].

Reliance on strength gains from self-weight consolidation just between installation and hook up or operation has been shown to provide potentially significant reductions in required shallow foundation footprints [7]. Prediction of evolving stiffness over the life of a monopile-supported wind turbine can provide more realistic predictions of through-life lateral displacements, better informing fatigue life predictions [8]; or better predictions of accumulated displacements or ratcheting of a taut moored suction pile anchor for a floating platform [9, 10].

By its definition, whole life design embraces the time-varying evolution of actions and resistances to create a continuous assessment of conditions of a structure—a ‘living design’, which can form a digital twin—i.e. a virtual model of a physical asset. The digital twin concept is gaining traction as an essential element of asset condition monitoring in offshore engineering (e.g. [11–13]). The concept of whole life geotechnical analysis is the best means to incorporate the foundation or anchoring system of an asset into its digital twin.

Whole life geotechnical design also has value in terms of assessing actual remaining design life and life extension feasibility. For example, comparing the observed response of a structure with the predicted response can help update estimated changes in soil response and refine the calculated remaining design life. Additionally, metocean conditions are increasingly recognized to vary through time due to climate change (e.g. [14, 15]). If a whole life geotechnical analysis shows that the geotechnical parameters and available capacity vary during this period, the two effects may offset each other. As time-varying design actions enter design practice, time-varying geotechnical responses can also be accommodated.

Whole life design also has value for late life and end of engineered life assessment of infrastructure, providing a basis to capture the changes in seabed resistance relevant to re-certification of structures due to new regulations, and providing information to support late life management and decommissioning options for offshore structures [1, 6].
Application to Different Types of Structures

Whole life geotechnical design can be applied in an offshore context to a range of shallow foundations—from large gravity bases to subsea mudmats; intermediate foundations—including monopiles and suction caissons; deep foundations—in compression or tension; a range of anchors; and long linear structures such as pipelines or cables. Some of these applications are illustrated later in the paper via the examples. Onshore, whole life design principles can be applied to a similarly broad range of infrastructure—but that is not the topic of this keynote paper, which has a focus on application of whole life geotechnical design for offshore engineering applications.

Application of Whole Life Geotechnical Design—Summary

Whole life geotechnical design leads to more appropriate and accurate design outcomes by considering the evolving geotechnical properties through life. Some soil conditions and applications lead to minimal whole life effects (e.g. highly overconsolidated clays), while in other cases (e.g. soft clays or highly mobile granular seabeds) whole life geotechnical responses can be significant. Whole life geotechnical design is a toolbox, not a single tool, and the approach must be tailored to the particular conditions of a specific design situation to yield greatest benefit.

Consolidation-Driven Changes for Whole Life Geotechnical Design

This paper will focus on consolidation-driven whole-life geotechnical response and design (Fig. 2a), with emphasis on evolving soil properties, including undrained strength, stiffness and coefficient of consolidation, from imposed actions followed by intervening consolidation. The fundamental concepts underpinning evolving soil properties from whole-life shearing and consolidation with demonstration of observed responses are set out followed by the theoretical basis for capturing evolution of soil properties from shearing and consolidation and predicting generation of excess pore pressures from cyclic loading. Then, analogs to extend the elemental frameworks to apply to boundary value problems to create whole-life geotechnical design frameworks are explored. Options for predicting geotechnical parameters for whole-life geotechnical design with particular mention of new laboratory-based developments are then discussed. Finally, closing comments on the application of whole-life geotechnical design in practice are presented.

Evolving Soil Properties from Whole Life Shearing Actions and Consolidation

Imposed actions across the life of a structure can be monotonic or cyclic and are often superimposed. Figure 1 presents a schematic of potential imposed actions across the design life of a geotechnical structure—from installation to decommissioning—and an indication of the evolving soil properties or profile, and subsequent resistance and stiffness, through life.

Fundamental soil mechanics principles describe the mechanisms through which undrained monotonic loading of soft clay leads to increases in excess pore pressure and a reduction in effective stress [16]; it is well established that undrained cyclic loading leads to softening due to build up of excess pore pressures [17]; and that consolidation, i.e. dissipation of excess pore pressures and reduction in void ratio, leads to increases in the undrained strength of a soil [18, 18]. In sequence, these principles form the basis of staged embankment or tailings dam construction [2, 3] as mentioned in Sect. 1, or more recently consideration of reuse of foundations onshore [20, 21].

A significant feature of offshore geotechnical engineering is that offshore structures are subjected to repeated actions, either from environmental loads (from wind, wave, currents, or in places ice loading) or from operational activities. Excess pore pressures generated from cyclic actions reduce effective stresses in the seabed and cause average and cyclic shear strains to develop with continued
cycling, ultimately leading to a loss of shear strength or stiffness of the seabed sediments.

Dissipation of excess pore pressures and plastic deformation of the soil matrix causes a change in void ratio; a reduction in void ratio for dissipation of positive excess pore pressures, i.e. consolidation, or an increase in void ratio for dissipation of negative excess pore pressures, i.e. swelling. Episodic shearing events—either monotonic or cyclic—with intervening consolidation periods have been shown to demonstrate periodic softening and hardening, with consolidation induced increases in strength often eclipsing shear induced softening in normally or lightly overconsolidated sediments [22–24, 26], and associated increases in stiffness and coefficient of consolidation [22, 23, 26, 27]. A selection of observations of the effects of repeated shearing and reconsolidation on strength, stiffness and coefficient of consolidation are highlighted in Figs. 3, 4, 5, 6. A threefold gain in resistance and interpreted undrained shear strength is observed from episodic T-bar tests in lightly overconsolidated kaolin in which a single penetration and extraction cycle was intervened with a period of rest to allow dissipation of excess pore pressures (Fig. 3a and b) [22, 23]. Figure 3c and d shows a similar response for a tolerably mobile shallow foundations undergoing episodic horizontal sliding with intervening consolidation [22, 23].

Figure 4 shows episodes of loss and then regain of strength from packets of undrained cyclic shearing interspersed with periods for reconsolidation in T-bar tests, shown both as magnitude of absolute shear strength and normalized as a degradation (or damage) factor [24]. Loss in strength is evident in each cyclic packet, although the extent of strength reduction reduces with increasing number of packets of cycling; with regain in strength due to consolidation back to (and potentially exceeding) the in situ strength. Figure 5 shows reducing settlement beneath a sliding foundation with each episode of monotonic shearing and consolidation, indicating increased stiffness [22], while Fig. 6 shows increasing coefficient of consolidation in a normally consolidated clay observed in centrifuge tests of a surface foundation undergoing packets of cycling with intervening consolidation. $c_v$ increases rapidly over the first few cycles of shear with intervening consolidation before stabilizing, indicating that after each dissipation stage, the soil under the foundation compresses, increasing stiffness, and hence coefficient of consolidation [25]. Beyond four packets of preloading and consolidation, the state of the soil has converged to an equilibrium position with no further volume changes—corresponding to a $c_v$ around tenfold greater than the initial value. Practically, this means that evolution of soil properties from later episodes of shear and consolidation will occur 10 times faster than predicted from the in situ $c_v$. Increases in $c_v$ have also been observed in direct simple shear element tests under repeated shear and consolidation [26] and are discussed later in the paper.

Changes in strength, stiffness and coefficient of consolidation are interdependent, linked by the effective stress-void ratio relationship (as illustrated in Fig. 7a) and unified in critical state soil mechanics [18]. The consolidation coefficient controls the rate at which consolidation-induced changes in soil properties will occur. Figure 7a shows the wide range of coefficient of consolidation for a clay at the same initial state due to differences in stiffness and permeability for a range of stress paths and drainage directions [27]. Figure 7b shows an example of the importance of capturing the evolving coefficient of consolidation to accurately identify changes in the ‘current’ resistance or stiffness response during soil-structure interaction. The figure shows the transition from undrained to drained sliding resistance as a function of dimensionless time factor $T = c_v t/d^2$ (where $d$ is drainage path length) showing a doubling of capacity between the two states [28].

Transitioning from observations of whole life geological response to a procedure for whole life geotechnical design relies on the geotechnical engineer having the tools to predict the necessary whole life changes in soil response or properties. In the case of consolidation-driven whole life design, these are tools need to predict generation of excess pore pressures that drive changes in effective stress, and changes in void ratio that drive changes in undrained strength, stiffness and coefficient of consolidation. The theoretical basis that enables predictive methods for consolidation-driven whole life design is outlined in the following section.

Theoretical Basis for Evolution of Geotechnical Parameters

Critical State Soil Mechanics (CSSM) for Capturing Unified Shear and Compression Response

Consolidation as a driver of using whole life geotechnical design can be quantified by the relationship between shear strength, effective stress and voids ratio of a geomaterial. This inter-relationship is underpinned by critical state soil mechanics (CSSM) [18, 19]. Figure 8 illustrates the critical state soil mechanics framework for considering the unified shear and compression response of a soil element, defined in three dimensional space of shear stress, $q$, effective stress, $p'$ and voids ratio, $e$.

The effective stress—voids ratio relationship is governed by the amount of drainage permitted during or following shear. Undrained shearing takes place at constant voids ratio, i.e. constant volume, with generation of excess...
Fig. 3 Increasing strength and resistance with episodic monotonic shearing and intervening consolidation a and b T-bar tests, c & d sliding foundation [22, 22]

Fig. 4 Reduction and subsequent increase in undrained shear strength with episodes of cyclic shearing and intervening consolidation [24]

Fig. 5 Increasing stiffness with episodic shearing and intervening consolidation [22]
pore pressures giving rise to a change in effective stress. In contrast, drained shearing is accompanied by a change in voids ratio and no generation of excess pore pressure. The shear stress–effective stress relationship is governed by the initial voids ratio of the geomaterial, more commonly described by its density or overconsolidation ratio. As a loosely packed granular geomaterial or normally or lightly overconsolidated clay (i.e. with a high initial voids ratio) will tend to contract during shear (i.e. reduce in volume, or voids ratio), undrained shear leads to generation of positive excess pore pressures, causing reduction in effective stress (O to A in Fig. 8); and drained shearing leads to a reduction in voids ratio, i.e. volume (O to B in Fig. 8). In contrast, as a densely packed granular geomaterial or heavily overconsolidated clay (i.e. with a low initial voids ratio) will tend to dilate during shear (i.e. increase in volume), undrained shear leads to generation of negative excess pore pressures, and increase in effective stress (O’ to A’ in Fig. 8) and drained shearing leads to an increase in voids ratio, i.e. volume (O’ to B’ in Fig. 8).

Dissipation of excess pore pressure following undrained shear follows a stress path along an unload-reload or ‘kappa’ line, eventually returning to the effective stress state prior to shear if sufficient time is allowed for all excess pore pressures to dissipate. The reduction in voids ratio being linked to the shear strength. Following critical state theory, a change in voids ratio due to shear and consolidation leads to a corresponding change in shear strength according to

\[ \Delta e = -\lambda \ln \left( \frac{s_{u,\text{cons}}}{s_u} \right) \]  

(3)

The ratio of consolidated to in situ (or previous) shear strength at the element level can then be expressed as

\[ \frac{s_{u,\text{cons}}}{s_u} = \exp \left( \frac{\Delta e}{\lambda} \right) \]  

(4)

Critical State Framework for Characterizing Response Under Episodic Monotonic Actions with Intervening Consolidation

The critical state framework can capture the stress, volume, strength relationship for a single cycle of (i.e. monotonic) shear stress (as shown in Fig. 8) or for multiple cycles comprising episodic monotonic shearing with intervening periods of consolidation (e.g. [23, 24]).

The evolution of shear strength due to episodic undrained shear with intervening consolidation and through drained shear is illustrated within the critical state soil mechanics framework in Fig. 9. The stress paths shown Fig. 9a relate to an initially normally consolidated deposit.
and loading conditions where each shearing episode leads to failure (i.e. reaches the critical state line, CSL), and each monotonic shearing episode is followed by full consolidation at the initial vertical effective stress. The stress conditions represent those directly beneath a tolerably mobile shallow foundation undergoing episodic horizontal sliding to failure [22]. For the shallow foundation application, vertical effective stress is constant and so it is convenient to use $\sigma'$ as the horizontal axis such that the stress state returns to this value following each consolidation stage.

For the normally consolidated conditions shown in Fig. 9, at a particular effective stress level, point O in Fig. 9a, undrained shearing to failure ($n = 1$) takes place at constant volume (voids ratio) leading to a reduction in effective stress until the stress state reaches the critical state (A). Subsequent consolidation leads to the stress state to travel along an unload/reload or ‘kappa’ line describing the relationship with volume change as excess pore pressure dissipates and the soil skeleton reconfigures to a new equilibrium under the applied action ($\sigma'$ in Fig. 9a). The lower voids ratio associated with point $O_1$ corresponds to a higher shear strength than at point O). The increase in strength with cycles ($n$) of shear and intervening consolidation is illustrated in Fig. 9b. The shear path may not reach the critical state line and excess pore pressures may not fully dissipate before a subsequent cycle of loading, but the same principles hold, as illustrated by the dashed line in Fig. 9a, and more fully illustrated in Fig. 10. The strength increase from cycles of undrained shear and intervening consolidation ultimately reaches the drained strength (B), where no further propensity for volume change exists.

From the triangular geometry of the stress-volume path, the undrained strength after n cycles of failure $s_{u,n}$ can be written as a multiple of the initial strength, $s_{u,1}$, for the case of full consolidation between each failure. This strength gain depends on the ratio between the drained and undrained strengths, $S$, (i.e. the stresses at points A and B) and for a stress state on the normal compression line, the ratio of the slopes of the kappa line and the critical state line, $k/k_c$:

$$\frac{s_{u,n}}{s_{u,1}} = S^{(1-(1-k)c)^n} \quad (5)$$

Other properties also evolve as a soil element follows a stress path involving episodes of shear stress and consolidation. Soil stiffness changes with effective stress and voids ratio and the consolidation coefficient is therefore also affected [27]. The stress history shown in Fig. 9 can also therefore serve as a useful proxy for the evolution of stiffness and permeability as well as strength. The general principles of evolving soil state and response with episodic monotonic actions and intervening consolidation described above are equally relevant to other boundary conditions. Similar responses have been demonstrated for episodic interface shear tests [29] and episodic T-bars [23].

An individual shearing event need not cause failure for the critical state framework to be useful (as shown in Fig. 10c and d). All stress paths that cause plastic strain
and hence expansion of the yield envelope eventually lead to the critical state line, which defines the critical voids ratio—effective stress and ultimate shear stress relationships. Importantly, the ultimate position on the critical state line is dependent only on the equilibrium effective stress of the sample and is independent of the initial voids ratio or stress history of the soil sample. Equally, full consolidation between shearing episodes is not required for application of a critical state framework (Fig. 10b and d) [1]. Pre-failure shearing or partial consolidation will require more cycles of episodic shear and consolidation to reach the CSL than for the case of episodes of shearing to failure and full consolidation, but ultimately will arrive at the same state.

**Shearing Inside the Yield Surface**

For episodic monotonic shear events that cause plastic strain at the current void ratio with each new shearing event, a basic critical state constitutive model, such as Cam Clay [18] or Modified Cam Clay [19], can capture the softening and hardening responses from generation and dissipation of excess pore pressures, respectively (as shown in Fig. 10). Pre-failure shearing response can also be captured, but only when additional plastic strain is generated in each subsequent cycle in order to continuously expand the yield surface. Therefore, the generation of excess pore pressure, indicated by the right to left traverse of the stress path in $\varepsilon \ln \sigma'$ space stress paths for the cases that do not reach the critical state line (i.e. Figure 10c and d), is only invoked once the stress path moves beyond the current yield surface. That is, a stress path, that traverses horizontally half way from its current position to the CSL in $\varepsilon \ln \sigma'$ space, lies half way between the current yield surface and the CSL in $\tau \sigma'$ space, i.e. very close to failure.

A range of pre-failure stress states may exist inside the yield envelope, but since, the basic Cam Clay models adopt an elastic response inside the yield envelope, excess pore pressure generation cannot be generated by repeated stress paths within the current yield envelope. As a result, softening and subsequent hardening, that can be invoked in reality by cyclic or episodic pre-failure shearing with intervening consolidation, cannot be captured when modelled with a basic CSSM model such as Cam Clay or Modified Cam Clay. Alternative approaches to capture excess pore pressure generation from pre-failure stress paths are explored in the following sections.
Extended Critical State Framework for Characterizing Undrained Cyclic Loading Response

It has long been recognized that cyclic loading causes softening of clay due to pore pressure generation [17] and that repeated loading usually causes the permanent strain to occur earlier than previous loading cycles. This implies that the yield stress limit is decreasing. In the conventional CSSM models such as Cam Clay and Modified Cam Clay, this cannot be reproduced as the yield surface is unaffected by activity in the elastic zone when the material is unloaded. Options for capturing these conditions are (i) a more complex constitutive model that can capture excess pore pressure generation pre-yield, or (2) an approximate method for predicting excess pore pressures to impose into a basic CSSM framework with a feedback loop. Both options are discussed in more detail below.

Clay Hypoplasticity

One option to capture pore pressure generation at pre-failure stress levels is adoption of a more complex constitutive model that does enable hardening under pre-yield stress paths. Hypoplasticity [30] is a potential alternative to elasto-plasticity, in which the yield surface is substituted with a state boundary surface that allows hardening within it by not distinguishing between elastic and plastic strains within the boundary surface envelope, such that hardening can occur at any level of strain (Fig. 11a). In hypoplasticity, the soil does not yield, instead it gradually approaches an asymptotic state. Clay hypoplasticity [31] is an advanced incrementally nonlinear constitutive model based on the general hypoplastic principles combined with traditional CSSM and developed specifically for simulation of fine-grained materials. An extended version of the model relying on the intergranular strain concept [32, 33] can capture more aspects of soil behaviour under loading reversals. This feature provides the capability of modelling cyclic loading behaviour and small strain stiffness effects to the model (Fig. 11b).

The downside of a more complex constitutive model is the additional parameters that need to be derived in order to apply the model. An alternative approach to capture the effect of excess pore pressure build up due to undrained pre-failure cyclic loading is to adopt a basic CSSM model and incorporate a feedback loop, updating the effective stress state with excess pore pressures (or strains) derived from an approximate method. An approximating expression for excess pore pressure, or ‘damage’ can be defined from a theoretical basis or observations [23]. Alternatively, the RSN accumulation procedure can be adopted to predict excess pore pressure (or strain). Geotechnical parameters can then be updated following consolidation prior to applying the subsequent packet of cyclic loading.

RSN Framework for Characterizing Pore Pressure and Strain Accumulation from Cyclic Loading

The RSN method is a well-established approximate method for predicting pore pressure or strain accumulation from cyclic loading using element test results from standard laboratory protocols [34, 35].

Rather than replicating a sequence of cyclic loading, e.g. a storm sequence, with high fidelity to estimate cyclic load response, a simple approach based on constant amplitude and frequency laboratory element tests is commonly adopted to determine necessary soil response and parameters. While amplitude and frequency of loading imposed to offshore infrastructure are irregular and the interaction is complex, cyclic load response is typically considered through constant amplitude and frequency laboratory element test data, direct simple shear or triaxial, as illustrated in Fig. 12 [36].

Laboratory element test results can subsequently be interpreted within a framework of so-called SN or RSN curves [34, 37], where R, S and N represent the mean stress, cyclic stress and number of cycles of stress, respectively. As such, SN curves are contours of cyclic shear stress (S) against number of cycles (N) required to generate a specified magnitude of shear strain or excess pore pressure, as illustrated in Fig. 13. Cyclic stress is typically normalized by consolidation stress (σ_u) or initial monotonous shear strength (s_u), while excess pore pressure ratio is defined by normalized consolidation stress (σ_u).

For contours of excess pore pressure generation, SN curves can be described (or fitted to laboratory test data) via Eqs. 3–11 [37]. One particular model for this, which is based on Verruijt’s pore pressure generation model [38], as extended to form S–N curves by others ([39, 39]). The model uses four constants (k_1 to k_4) to define a single relationship between the generated excess pore pressure ratio—referred to as damage, D, normalized shear stress, S, and number of cycles, N.

\[
D = \frac{\Delta \mu}{\sigma_u} = k_1 \left(1 - e^{-k_2 N \left(\frac{\tau_{cyc}}{\tau_{mono}} - k_3 \right)^{-k_4}}\right) \quad (6)
\]

\[
S = \frac{\tau_{cyc}}{s_{u,mono}} = \left(\frac{\ln \left(1 - \frac{\Delta \mu}{\sigma_u} \right)}{-k_2 N} + k_4\right)^{1/k_3} \quad (7)
\]

\[
N = \frac{1 - \frac{\Delta \mu}{\sigma_u}}{-k_2 \left(\frac{\tau_{cyc}}{\tau_{mono}} - k_3 \right)^{k_4}} \quad \text{or} \quad N = \frac{\ln \left(1 - \frac{\Delta \mu}{\sigma_u} \right)}{-k_2 (S - k_4)} \quad (8)
\]
Fig. 11 Hypoplasticity as an option to capture hardening under pre-yield stress paths. 
(a) illustration of the state boundary surface [32], and 
(b) effect of intergranular strain concept capturing changes in void ratio at small strain [33].

Fig. 12 Characterization of cyclic loading. 
(a) Irregular and regular amplitude and frequency, 
(b) symmetric and unsymmetric modes of cyclic loading, and 
(c) shear strain and excess pore pressure development during unsymmetric cyclic loading [36].

Fig. 13 Cyclic strain and excess pore pressure contour diagrams for symmetric cyclic loading. ‘S–N’ curves [36].
where \( k_1 \) represents the maximum damage that can occur \( \left( k_1 = D_{\text{max}} = \frac{\Delta u_{\text{max}}}{\Delta \sigma} = 1 \right) \). \( k_2 \) and \( k_3 \) are empirical constants controlling the rate of pore pressure generation, and \( k_4 \) represents the lower asymptotic damage boundary or \( D_{\min} \) where \( \{ \ldots \} \) represents Macaulay brackets such that no damage is accumulated for \( \frac{\Delta u}{k_{\Delta \sigma}} < k_4 \).

The effect of imposed actions offshore having varying amplitude is re-incorporated in an idealized fashion into the analysis through an accumulation procedure. As an alternative to modelling the complete loading sequence, an equivalent number of cycles of the peak load is used to represent the excess pore pressure or strain that would be accumulated during a loading sequence, e.g. from a storm of many waves of varying intensity (as stress intensity starts small and increases as the storm builds to a maximum before decreasing as the storm passes). The accumulation procedure starts by idealizing a loading, or storm, sequence into packets of load (bins) of increasing magnitude and estimating the effective damage, in terms of excess pore pressure or strain, generated for the number of load events in each bin. Each packet of loading, starting with the smallest and most frequent shear stress, is then plotted on the excess pore pressure (or strain) contour plot, at the relevant stress level for the relevant number of cycles of load. The notional contour for that magnitude of excess pore pressure is then traced back (parallel to the closest actual contour) to reach the next magnitude of shear stress in the idealized storm. The process is repeated for each stress level, by creating a horizontal line at the stress level for the corresponding number of cycles, thus accumulating the damage in each bin. The process finishes at the (single) peak design load, and the corresponding number of cycles on the SN curve represents the equivalent number of cycles \( N_{\text{eq}} \) of the peak shear stress that would cause the same excess pore pressure generation or strain as the entire storm sequence (Fig. 14). The accumulation method enables the damage from a general spectrum of loads to be reduced to a single scalar quantity as excess pore pressure or strain. The accumulation procedure is well established and routinely used in offshore geotechnical design and is similar to liquefaction assessment methods in geotechnical earthquake design and structural fatigue modelling.

SN curves, as illustrated above, can only capture the cyclic load response to a single mode of cyclic loading and is typically used to describe the response to symmetric cyclic loading, i.e. two-way cycling about zero average shear stress. In reality, stress states in the soil beneath structures comprise static and cyclic loads. The effect of the level of average shear stress on the cyclic load response can be considered with a third axis, ‘\( R \)’ (Fig. 15). Considering the 2D slice of the RSN plot shown in Fig. 15a, the excess pore pressure ratio that would be reached at 10 equivalent cycles (of the peak load) for any given average and cyclic shear stress can be found. For example, an average and cyclic imposed shear stress of 0.5\( s_u \) would lead to a pore pressure ratio of around 0.2, while increasing the cyclic shear stress to 0.6\( s_u \), keeping the average shear stress constant would lead to a pore pressure ratio of 0.3. SN and RSN curves are an established approach for predicting soil softening from build-up of excess pore pressure during undrained cyclic loading that can be coupled with a basic CSSM model to capture the effects of undrained cyclic actions with intervening periods of consolidation.

In the following section, analogs for expanding the soil-element level concepts to a whole geotechnical system (via a ‘macro model’) are presented, which enable prediction of the whole life response of fine grained soils for a range of boundary value problems and loading conditions.

### Macro-Models to Capture Whole Life Response

#### Introduction

The critical state framework was developed for and is most commonly applied at element level. For application to boundary value problems, critical state constitutive models are used in conjunction with finite element analysis. However, such an approach requires bespoke model creation for ad hoc analyses for a given design condition. Alternatively, results from a suite of parametric finite element analyses with a critical state constitutive model, or results from scaled model or field testing can be used to validate a generalized analytical macro-model based on CSSM principles.

In the following sections, analogs are presented that (1) recast soil elements as zones within a boundary value problem (‘lumped’ or ‘layered’) and (2) recast changes in effective stress driving softening and consolidation, as damage and hardening indices. The former approach is well suited to boundary value problems where the total stress state is known or can be reasonably assumed, e.g. beneath a near surface structure, such as a shallow foundation or pipeline. In this case, fundamental critical state relationships can be applied relatively directly with factors to capture the variation in effective stress and volume change in the prescribed zone. The latter approach is well suited to boundary value problems where the total stress state is less easily defined, for example for embedded structures such as piles, caissons or plate anchors.
‘Lumped’ and ‘Layered’ Systems; Spring and Slider Analogs

Simplified analytical models applying critical state soil mechanics principles to ‘lumps’ or ‘layers’ of soil can provide an elegant generalized solution that is quick and easy to apply for preliminary design without the need for additional finite element analyses and can be incorporated into a wider system model involving the structure. Figure 16 illustrates the different modelling concepts and a simple spring-slider analogy for each system [1].

A single element of soil within a continuum (Fig. 16a) can be represented by a single spring and slider to represent stiffness and strength, with stress or strain as input at both ends of the system. A finite element analysis approach simply replicates this system in each element of the mesh and in all coordinate directions being considered, with the spring-siders replaced by the adopted critical state constitutive model (Fig. 16b). A ‘lumped element’ approach (Fig. 16c) applies the essential critical state framework to a region of soil, modified by a scaling factor, or ‘lumped factor’ to account for the non-uniform distribution of stress and void ratio change over the region governing changes in strength, stiffness and coefficient of consolidation. This can be represented by a single spring and slider as for a single element (or a family of springs and sliders known as Iwan elements [42]), but with stress or displacement as input to the system and a fixed boundary at the other end. The general lumped element (or macro-element) approach has been applied for a range of boundary value problems including shallow foundations [43–45], pipelines [46, 47] and penetrometers [24]. The lumped element can be described as a formal constitutive model (e.g. [48, 49]) or as uncoupled relationships between applied load and displacement in each direction (e.g. [50, 51]). These relationships can themselves include time-dependent changes in strength following critical state principles (e.g. [24, 43, 45–47]).

A ‘layered element’ approach (Fig. 16d) applies the basic critical state framework to soil layers to enable variations in soil properties with depth (or stress level) to be considered discretely and then summed—analogous to the oedometer method of predicting settlement. This approach can be illustrated as a series of spring-slider systems in sequence. As with the lumped element model, shear stress or displacement is input to the system with a fixed boundary at the other end. The decay of stress with depth can be estimated using elastic solutions. The layered model is illustrated in Fig. 16d with a series of springs and sliders, and each of these can be defined using a critical state approach, so that their stiffness (spring) and strength (slider) can evolve with time and loading history.

The following table shows the results of the simplified analytical models:

| Number | h/h<sub>max</sub> | τ/σ<sub>′vc</sub> |
|--------|------------------|--------------------|
| 1      | 100              | 0.200              |
| 2      | 95               | 0.190              |
| 4      | 86               | 0.172              |
| 15     | 70               | 0.140              |
| 30     | 61               | 0.122              |
| 50     | 49               | 0.098              |
| 400    | 40               | 0.080              |
| 700    | 33               | 0.066              |

Fig. 14 Excess pore pressure accumulation procedure – packets of stresses for an idealized storm (see table) and pore pressure accumulation procedure on a contour diagram created from constant amplitude laboratory element test results [36]

Fig. 15 Excess pore pressure contour diagrams for unsymmetric cyclic loading, ‘RSN’ curve a a 2D slice for N = 10 for NC Drammen Clay [36] after [17], b a 3D RSN curve showing accumulation procedure [41]
layered approach has recently been applied to frameworks for predicting changes in resistance and settlement of tolerably mobile subsea foundations [23, 52, 53].

The whole life principle of time-varying resistance is represented in any of these models through the changing spring stiffness and slider resistance with time, with those changes being efficiently described via critical state principles.

**Example Application—Lumped Element Approach**

**Consolidated Undrained Capacity of Shallow Foundations** The challenge in linking strength changes at an element level with foundation capacity is that the increase in shear strength due to self-weight loading is non-uniform across the zone of influence and is related to the stress change within the pressure bulb developed in the soil due to the application of the foundation load or preload. Figure 17 shows the initial excess pore pressure bulb around a circular surface plate and various shallowly embedded foundations [54], and Fig. 18 shows the change in shear strength of a normally consolidated deposit under vertical (self-weight) loading of a circular surface foundation followed by full consolidation [45], demonstrating the variation in pore pressure and strength across the affected zone (or lump). Contours of the ratio of the consolidated undrained strength to the initial undrained shear strength in the virgin soil show a doubling of the initial undrained strength is achieved close to the underside of the foundation, decreasing with distance from the foundation. The question is then, how to determine the operative shear strength that will govern foundation performance under the subsequent load path?

Results from a suite of finite element analyses investigating the effect of relative preload (preload/initial undrained capacity, $V_p/V_u$), degree of consolidation (T), and over consolidation ratio (OCR), showed consolidated undrained vertical bearing capacity of strip and circular surface foundations can be predicted by a simple lumped element critical state model [43]. The basis of the framework is illustrated graphically in Fig. 19a. The applied stress can be divided into elastic and plastic stress increments depending on the over consolidation ratio of the in situ soil. These stresses are then converted to an ‘operative’ preload defined by the applied preload scaled by a stress factor $f_r$, to account for the non-uniform distribution of stress beneath the foundation (and to account for the vertical rather than mean stress being considered). The operative shear strength, $s_u,op$, can then be defined from the change in voids ratio, $\Delta e$, under the operative preload, adjusted by a constant shear strength factor, $f_{su}$, to account for the non-uniform distribution of the increase in shear strength in the zone of soil that controls the consolidated capacity. The stress factor affects the level of preload at which the rate of gain in capacity increases (i.e. it controls the position of the change in gradient of the curves in Fig. 19b) while the shear strength factor controls the rate of gain in capacity with preload (i.e. the gradient of the curves in Fig. 19). For normally consolidated conditions, there is effectively a single scaling parameter $f_r f_{su}$ for a given load path.

For normally consolidated conditions, in which all of the applied stress causes plastic compression of the foundation soil, the gain in undrained vertical bearing capacity due to self-weight loading and full consolidation can be given by
where \( V_{u,\text{cons\_max}} \) is the maximum undrained vertical bearing capacity, i.e. after full consolidation; \( V_u \) is the unconsolidated undrained vertical bearing capacity; \( f_r \) and \( f_{su} \) are the stress and strength factors as described above; \( R \) is the normally consolidated undrained strength ratio, \( s_u / \sigma_v' \); \( V_p \) is the applied preload (i.e. self-weight load) and \( N_{cv} \) is the vertical bearing capacity factor for undrained unconsolidated conditions. \( N_{cv} \) can be determined from either classical bearing capacity theory or from numerical and analytical solutions for various boundary conditions (e.g. [55–59]).

The study also showed that consolidated undrained capacity gain could be predicted through a hardening law, linking gain in capacity to settlement as a result of reduction in voids ratio, as a linear function of the degree of consolidation (Fig. 20).

The lumped element critical state inspired model developed for surface strip and circular foundations outlined above [43] has been extended to multi-directional capacity following vertical preloading and applied to rectangular foundation geometry [44], skirted circular foundations [45].

Figure 21 illustrates the multi-directional extension of the framework for the case of rectangular mudmats, showing the gain in uniaxial capacity under actions in each of six degrees of freedom can be described as a linear function of vertical preload (\( V_p/V_u \)) after the method presented by Gourvenec et al. [43]. Vertical, horizontal, moment or torsional capacity can be defined by variants of Eq. 6, with scaling factors according to the respective load path, as shown in Fig. 21 for a rectangular shallow foundation on normally consolidated clay.

The study also showed the unconsolidated undrained failure envelope can be scaled by both vertical preload and degree of consolidation to give consolidated undrained capacity [44]. Figure 22 shows potential efficiency in foundation footprint by relying on strength gains from self-weight consolidation from predictions with the lumped element macro-model [7].
induced vertical preloading and consolidation [46], most recently incorporating the effects of large displacement during installation [47]. The macro-model was calibrated against a suite of parametric finite element analyses [46, 47] using a Modified Cam Clay constitutive model. Figure 23 shows the potential increased lateral break out resistance with increasing levels of preload and consolidation (difference between the inner pink yield envelope and the outer envelopes).

**Summary** These analytical lumped element critical state models enable consolidated undrained capacity to be predicted for a range of seabed and structure conditions without the need for bespoke finite element analysis. These models provide results essentially instantaneously and are a valuable tool for preliminary design, sizing or parametric analysis where numerous ad hoc finite element analyses might otherwise be required. Information on applying these macro-models in routine design is presented later in the paper. The examples shown here demonstrate strength and capacity gains by relying on self-weight installation and consolidation. The method can be equally applied to episodic cyclic loading with intervening consolidation.

| Uniaxial capacity | $f_{\text{feq}}$ |
|-------------------|-----------------|
| $V_{\text{ul,cons \_max}}$ | 0.439 |
| $H_{\text{xx,cons \_max}}$ | 0.919 |
| $H_{\text{yy,cons \_max}}$ | 0.919 |
| $M_{\text{xx,cons \_max}}$ | 0.345 |
| $M_{\text{yy,cons \_max}}$ | 0.538 |
| $T_{\text{xx,cons \_max}}$ | 1.071 |

![Fig. 20](Fig_20.png) Hardening rule for prediction of gains in consolidated undrained capacity from observed settlements a for full consolidation and b for partial consolidation [after 43]

![Fig. 21](Fig_21.png) Consolidated undrained multidirectional capacity

![Fig. 22](Fig_22.png) Predicted reduction in footprint area due to consolidation strength gains from lumped element macro-model
Example Application—Layered Element Approach

The layered element approach is similar to the lumped approach but enables changes in stress and strength with depth to be derived.

Evolution of Sliding Resistance and Accumulated Displacement of a Tolerably Mobile Foundation  A generalised layered element critical state framework has been proposed for the whole life response of a sliding shallow foundation on soft clay, validated against centrifuge modelling [22, 23]. The design philosophy of a tolerably mobile mudmat is that it would slide across the seabed, tolerably, in response to the operational loads resulting from the thermal expansion and contraction of the attached pipelines, rather than being designed sufficiently large to resist all the applied loading and remain stationary. Tolerably mobile foundations can be designed with a smaller footprint than a stationary foundation, easing installation challenges of subsea mudmats as developments move into areas with softer seabeds, higher operational temperatures or heavier structures are required as more functions are carried out by the subsea structures that the foundations support. An example of a tolerably mobile subsea mudmat is illustrated schematically in Fig. 24. During sliding, shear-induced pore pressures are generated in the supporting seabed, which then dissipate, while the foundation is stationary in the operational position. The reduction in void ratio during consolidation is directly linked to the increased sliding resistance and incremental settlement from the preceding shear episode.

The simple 1D boundary conditions underpinning the layered element macro-model for the sliding foundation is illustrated in Fig. 25a, and the critical state interpretation for the model is as shown in Fig. 9 and described in the accompanying text for episodic shear with intervening consolidation. Figure 25b–d shows the macro-model predictions against observations from centrifuge modelling [22] of evolution of undrained shear strength with depth, sliding resistance and settlement from a lifetime of episodes of sliding and consolidation.

Summary Layered element critical state models, similar to the lumped element macro-models outlined before, enable predicted response for a range of seabed and structure conditions, without the need for bespoke finite element analysis, and near instantaneous results from the simple calculation tool. Layered models have the further benefit of predicting settlement from cumulative changes in void ratio, as in the classical ‘oedometer method’ for purely vertical load.

Damage and Hardening Analogue

CSSM can be directly applied to lumped or layered zones to create a macro-model for a boundary value problem if a representative stress state can be reasonably estimated. This is relatively straightforward for surface or near surface infrastructure. For example, beneath a surface foundation, the total stress state can be reasonably estimated as the self-weight or gravity loads of the foundation and what it supports, plus geostatic stresses with depth. For embedded structures, and particularly those subjected to lateral actions, the representative stress state is not straightforward to define, and hence deriving the effective stress state to define changes in void ratio is not possible. In this
situation, a critical state inspired (CSI) approach can be adopted via proxies of damage (or softening) and hardening to represent excess pore pressure generation and change in void ratio, respectively, in an extension to the Veruijt model [38].

A damage and hardening analogue CSI macro-model has recently been developed for soil initially on the wet side of the critical state line, such that positive excess pore pressures will be generated on shearing as the soil exhibits a tendency to contract, and subsequent dissipation leads to hardening and increases in shear strength and stiffness [60] (Fig. 26). For these conditions, undrained cyclic loading from the initial state generates positive excess pore pressures, captured by a proxy parameter, the damage index, \( D \) (\( 0 < D < 1 \)). Consolidation causes densification through dissipation of excess pore pressure and a reduction in void ratio, which is captured by a time-dependent reduction in the damage index and increase in a so-called hardening index, \( H \).

Damage can be defined by different methods depending on the boundary value problem of interest and whether imposed actions are load or displacement controlled, and therefore, if damage is defined by excess pore pressure or strain. A RSN damage accumulation method can be used when imposed actions are load controlled and excess pore pressure accumulation can be directly calculated. Alternatively, an approximating expression linking excess pore pressure to mobilized shear stress can be adopted [23]. For strain controlled actions, an approximating expression, or a function of damage against strain, can be derived from observations (either field, model test or numerical) [60]. The RSN approach and approximate strain-controlled damage function applied to damage definition are illustrated in the later examples. Consolidation causes densification through dissipation of excess pore pressure and reduction in void ratio and can be captured by a concurrent time-dependent reduction in the damage index and increase in a so-called hardening index, \( H \). The process being analogous to consolidation along an unload-reload or ‘kappa’ line in traditional CSSM is related by a parameter called \( k^* \). In the initial state, \( H = 0 \), and the minimum strength \( s_{u,f} = s_{u,i}/S_{0} \), where \( S_{0} \) is the initial soil sensitivity. As the soil progressively densifies, the sensitivity reduces to unity and the strength converges towards a maximum or final strength \( s_{u, f} \), which is also linked to the potential change in soil shear strength from densification. Soil strength for a particular hardening value is bracketed by initial and remoulded values of strengths [such as in 24] in contrast to a unique (critical state) strength for a given hardening level (or void ratio). This feature enables the model to capture remoulding and recovery of strength.

Figure 26 shows cases A, B and C that represent three cases of continuous cyclic loading at different rates relative to the consolidation process. Case A represents ‘fast’
cycling with negligible consolidation such that the strength simply falls from the initial to the remoulded strength value. Case C represents conditions where a high level of dissipation takes place either during or in between cyclic actions, for example where cycling is ‘slow’ or where an undrained shear event is followed by a period of consolidation. In this case, the effect of hardening eclipses the damage (or softening) and the strength rises towards the final, maximum limit, \( s_u \), with every cycle. Case B represents continuous cycling at a rate between Case A and Case C. Case E represents episodes of fast cyclic loading interspersed with consolidation such that softening during each cyclic loading period is followed by hardening as the effect of pore pressure dissipation eclipses the effect of generation. With each packet of cyclic loading and consolidation, strength and stiffness increase and sensitivity reduces.

**Examples of Damage and Hardening Critical State Inspired Model**

**Stiffness Evolution Around a Pile Under Lateral Cyclic Actions** The CSI model outlined above has been applied to capture stiffness and strength evolution around a pile in soft clay under displacement controlled lateral cyclic action [60]. Damage is defined by an approximating expression of damage vs scaled displacement fitted to a database of results from model pile tests and cyclic T-bar tests. The CSI model is coupled with a parallel Iwan (PI) model [61] for the hysteretic non-linearity of the p-y response. The resulting model is thus called ‘PICSI’ [60].

A two-component model for the variation in damage combines accumulated lateral movement of the pile, \( \delta y \), and time-dependent decay through a dissipation process linked to \( c_v \):

\[
\delta D = d_r (1 - D) \delta y \left| \frac{\delta y}{\delta t} \right| \frac{\delta y}{\delta t} - c_v \left( \frac{\delta y}{\delta t} \right) D^2 \delta t
\]  

(10)

The full set of model parameters are described by the authors [60]. The dissipation of damage leads to concurrent hardening, via a function of the form:

\[
\frac{\delta H}{\delta t} = c_r (1 - H)^{b_p} \frac{\left( \kappa \kappa \right)}{D^2} D^2 \delta t
\]  

(11)

So that the rate of hardening is proportional to the current damage level and reduces to zero as the hardening parameter approaches unity, as illustrated by the paths in Fig. 26. The specific form of these expressions may require modification for different boundary value problems, but the general concepts set out in Fig. 26 meet the whole life design requirement of capturing the contrasting and connected effects of softening from damage and subsequent hardening.

The PICSI model is compared with centrifuge test results of a pile embedded in a carbonate silt subjected to displacement controlled laterally cycling. 10,000 cycles in total were imposed. During the first 500 cycles, softening and reduction in stiffness to around 7% of the initial secant stiffness were observed. This was followed by recovery with a stiffness at the end of the test approximately two times the minimum value. Figure 27 shows the reduction and recovery of stiffness—or the damage and hardening—observed in the centrifuge test and predicted by the critical state inspired macro-model [60]. The response shown in Fig. 27 represents ‘Case B’ shown in Fig. 26.

**Changing Capacity of Embedded Plate Anchors Due to Cyclic Actions** The CSI damage and hardening analogue model has also been applied to capture changes in capacity of an embedded plate anchor subjected to cyclic loading [41]. The application case considers a wave buoy moored by an embedded plate anchor. The CSI model updates current undrained strength \( s_u \) relative to the initial value, \( s_{u0} \), depending on the hardening and damage indices. The updated undrained shear strength can then be directly applied to the anchor bearing capacity calculation.
Figure 28 shows that in this case, the imposed actions to the anchor are a time series of loading based on seastate, and as such damage is cast in terms of pore pressure generation (rather than strain). The damage index was calculated by an extension of the Verruijt model for pore pressure build up [37] and the pore pressure accumulation procedure, in which damage per cycle is controlled by both the mean and cyclic shear stress. Hardening was defined via the CSI model outlined above. The analysis shows an initial drop in undrained strength at the beginning from damage caused by cyclic loading, followed by subsequent recovery and increase in shear strength.

**Deriving Whole Life Geotechnical Design Parameters**

**Necessary Parameters**

Application of whole life geotechnical design requires prediction of the change in seafloor and seabed properties due to design actions over the whole life of a structure as input into any developed design calculation. Conventional current site investigation practice for clay seabeds focusses on quantifying the intact, cyclic undrained and remoulded soil properties via a combination of well-established in situ and laboratory testing. Triaxial and simple shear element testing protocols are predominantly used [17, 34, 37] and flow round penetrometers, such as cyclic T-bar testing both in the field and in a geotechnical centrifuge environment [62–64]. Extension of this testing to the changing undrained strength through episodes of shearing (to failure or pre-failure) and consolidation is required to support whole life design on soft soils.

**In Situ or Model Test Determination**

Cyclic T-bar penetrometer tests have been most commonly adopted to characterize softening and hardening responses in situ or in model tests. Examples of continuous undrained and episodic T-bar tests in centrifuge model tests are illustrated in Figs. 3 and 4 exhibiting both softening and progressive hardening. A lumped element critical state framework has been developed for cyclic T-bar response and been shown to predict the observed response well [24]. Further, the softening (or damage) and hardening characteristics from T-bar penetrometer tests have been shown to conform to lumped element critical state models for other boundary value problems, such as the sliding foundation [22]. Despite these attractions, there are barriers to adoption of T-bar testing to characterize whole life properties for routine design. Retrieval of seabed samples in sufficient volume to be used for centrifuge modelling of episodic flow round penetrometer testing is impractical for routine characterization. Laboratory or on-deck miniature T-bar tests at 1 g can be performed in box core samples [65]. However, only shallow soils can be tested in this way, and the consolidation times in episodic tests can remain long, despite miniaturization of the penetrometer. Carrying out episodic T-bar tests at the seafloor with current vessel-supported site investigation methods is also impractical due to the time required for the dissipation phases. However, this limitation could be overcome in the future with remotely operated or autonomous penetrometer systems.

For very near-surface soils, new shallow penetrometer tools including the toroid and hemiball have been developed to determine interface cyclic hardening characteristics. These devices require relatively short drainage times due to the proximity of the surface boundary. Shallow penetrometers have been inspired by the needs of surface-laid pipeline design, for which episodic hardening can be a key parameter. Versions of these penetrometers scaled for use in box cores have been successfully trialled within the RIGSS JIP (www.rigssjip.com) [66, 67].

**Element Test-Based Determination**

Adoption of laboratory element tests to characterize episodic cyclic hardening has some obvious advantages over penetrometer tests, namely that a single element under uniform stresses is tested, and a smaller volume of material is required. Episodic interface shear box tests have been shown to characterize the evolution of pipeline-seabed friction on soft soils and a critical state-based solution has been developed for the rate at which excess pore pressure generated during shearing will dissipate for these conditions [68]. Direct simple shear (DSS) has also been used to examine cumulative settlements and the resulting strength...
gain from dissipation of storm-induced cyclic pore pressure, with tests involving a single packet of pre-failure cycles followed by drainage, and then shearing to failure [69, 70]. Recent research has extended this concept to consider multiple consolidation phases within a DSS test, to follow the type of episodic stress path outlined in Fig. 10c [26]. Figure 29 shows the observed cyclic hardening as a function of cycles of episodic shear and consolidation, in terms of evolving undrained shear strength and coefficient of consolidation. The observed changing shear strength is compared with predictions from three different methods, based on measured changes in void ratio [18], measured pore pressure changes [72], and an assumed effective stress path that varies with OCR [70]. A lumped element critical state framework considering pre-failure shearing and either full or partial consolidation between cycles of load, similar to as shown in Fig. 10c and d, has been shown to replicate the observed response.

More recent work has considered packets of pre-failure cyclic loading with intervening consolidation of a soft natural clay in DSS and the effect on the soil response as a function of the number of cycles per packet and number of packets of loading [71]. In the tests, pre-failure cyclic loading is applied in packets of \( n = 2, 4, 10 \) and 20 cycles, with varying number of packets of cyclic loading, \( N \), with intervening consolidation. Figure 30 shows some preliminary results from this current work.

Figure 30a shows the shear stress–shear strain path in the first and 5th packet, i.e. \( N = 1 \) and 5, for \( n = 10 \) cycles applied in a single packet, demonstrating softening within the packet due to the undrained cycling, alongside the hardening between packets due to the intervening consolidation. In this example, considerably greater stiffness is observed in the last packet compared with the first packet. Figure 30b shows the generation of excess pore pressure during the first and 5th packet for the same test phases, i.e.
$N = 1$ and 5 for $n = 10$ cycles per packet, clearly showing the reducing tendency for excess pore pressure generation with increasing packets of cycling with intervening consolidation. Reduced tendency for excess pore pressure generation in subsequent packets of loading, and therefore potential for consolidation in the intervening periods, leads to reducing volumetric strain, and hence void ratio with each packet. In turn, these changes in void ratio govern changes in strength, stiffness and coefficient of consolidation in each packet of loading.

Figure Fig. 30c and d show comparisons of accumulated damage (or excess pore pressure generation) and change in void ratio after 20 cycles of undrained pre-failure shear, applied as N packets of n cycles of loading followed by consolidation. Figure 30c shows that a greater number of cycles per packet lead to greater damage, but greater potential for hardening once consolidation takes place (Fig. 30d).

Direct simple shear (DSS) is well suited to episodic testing, more so than triaxial testing, since the small sample size leads to faster equilibration of effective stresses following shearing, and of course many fold faster than penetrometer testing. For example, the test results shown in Fig. 29 were derived from 100 mm diameter, 25 mm
height samples of normally consolidated kaolin clay and reached equilibrium within 30 min of a shear episode. Observations so far indicate that DSS testing has considerable potential as a method to characterize the necessary whole life geotechnical properties for whole life geotechnical design to be adopted in routine engineering practice.

Applying Whole Life Design in Practice

The examples in this paper have illustrated potential design benefits of adopting a whole life geotechnical design approach. In order for the approach to achieve widespread uptake, new methodologies are needed to derive the necessary design parameters and accessible design tools are needed for engineering practice. Open dissemination of the methodologies and tools to predict the changing geotechnical conditions in response to a whole life of actions will encourage adoption, acceptance and consistency. Open and accessible tools that support this process and which see frequent use by industry include www.webappsforengineers.com, established and maintained by the author, and www.geocalcs.com, established and maintained by Dr James Doherty, UWA.

Concluding Remarks

This paper has presented theory and applications of whole life geotechnical design to offshore geotechnics—an approach which seeks to predict soil-structure response across the design life by considering the whole life of imposed actions coupled with geotechnical properties that evolve with each action. The overarching philosophy of whole life geotechnical design has been set out along with examples where whole life design principles are routinely adopted in geotechnical engineering and beyond. The breadth of potential applications to offshore geotechnical applications has been introduced, before focusing on one particular driver for whole life geotechnical design, consolidation. The theory underpinning the evolution of strength and stiffness changes—both softening and hardening, has been set out alongside illustrations of observed whole life evolution of strength, stiffness and coefficient of consolidation. Theoretical frameworks extending the elemental response of soil to whole life actions to boundary value problems have then been introduced. Two distinct macro-modelling approaches were presented (i) a lumped or layered element and spring-slider analogue based on critical state principles and (ii) a critical state inspired analogue adopting proxies of damage and hardening in place of excess pore pressure generation and void ratio change due to dissipation. Derivation of the necessary parameters to calibrate and use whole life design models in geotechnical engineering has then been touched on, demonstrating promising new findings from cyclic episodic DSS element testing. In closing, the necessity for, and examples of, accessible design tools to uptake of whole life geotechnical design has been set out.

In the offshore geotechnical context, the emerging philosophy of whole life geotechnical design offers immense potential to support the accelerating offshore renewables sector. Design efficiencies are increasingly essential for developments containing tens or hundreds of offshore wind turbines for a similar energy yield of a single hydrocarbon development. The insights enabled by considering the soil-structure response across the design life may permit smaller foundations and anchors, easing installation and cost (both direct and embodied carbon) to meet the required factor of safety compared to a traditional offshore geotechnical design approach. Insights into the evolving stiffness through life from lateral displacements of monopile supported wind turbines can better inform fatigue life predictions; or better predict accumulated displacements or ratcheting of a taut moored anchor piles for a floating wind turbine to enable adequate but efficient designs. Whole life geotechnical design approaches also have immense value during and beyond the design life. Whole life design enables a ‘living design’ or ‘digital twin’, a virtual model of a physical asset, for observations of performance after events or episodes to be compared against those predicted at the initial design, and forecast response to be updated based on those new insights. This measurement of performance through life enables better prediction of the actual remaining design life, to inform on late life management and decommissioning, whether required vessel capacity to remove infrastructure or stability of infrastructure remaining in situ beyond the operational design life. Observations and learning from living designs also provide feedback loops to continuously improve methodologies to predict parameters for evolving soil-structure response and calculation tools for initial design. Efficiency and optimization of geotechnical design are essential to enable the offshore renewables transition necessary to support decarbonisation of the global energy sector and whole life geotechnical design has an important role to play.

Acknowledgements The Author’s position is supported by the Royal Academy of Engineering under the Chairs in Emerging Technologies scheme. The Author thanks the Royal Academy of Engineering for their support and the team at the Centre of Excellence for the Chair in Emerging Technologies in Intelligent & Resilient Ocean Engineering (IROE) and acknowledges those in particular whose work is highlighted in this paper.

Funding The Author’s position is supported by the Royal Academy of Engineering under the Chairs in Emerging Technologies scheme.
Declarations

Conflict of interest  The Author declares they have no conflict of interest.

Open Access  This article is licensed under a Creative Commons Attribution 4.0 International License, which permits use, sharing, adaptation, distribution and reproduction in any medium or format, as long as you give appropriate credit to the original author(s) and the source, provide a link to the Creative Commons licence, and indicate if changes were made. The images or other third party material in this article are included in the article’s Creative Commons licence, unless indicated otherwise in a credit line to the material. If material is not included in the article’s Creative Commons licence and your intended use is not permitted by statutory regulation or exceeds the permitted use, you will need to obtain permission directly from the copyright holder. To view a copy of this licence, visit http://creativecommons.org/licenses/by/4.0/.

References

1. Gourvenec S (2020) Whole-life geotechnical design: What is it? What’s it for? So what? And what next? Proc. 4th International Symposium on Frontiers in Offshore Geotechnics. Austin, Texas, USA, ASCE Geo-Institute and DFI
2. Ladd CC (1991) Stability evaluation during staged construction (22nd Terzaghi Lecture). ASCE J of Geotech Eng 117(4):540–615
3. Mittal HK, Morgenstern NR (1975) Parameters for the design of tailings dams. Can Geotech J 12:235–261
4. Tjelta TI (1993) Foundation behaviour of Gulffaks C. Proc. Offshore Site Investigation and Fdn Behaviour. Soc for Underwater Tech 28:451–467
5. Tom JR, Draper SD, White DJ, O’Neill MP (2016) Risk-Based Assessment of Scour Around Subsea Infrastructure. Proc. Offshore Technology Conference, Houston OTC27131
6. Gourvenec S and White DJ (2017) In situ decommissioning of subsea infrastructure. Proc. Conf. Offshore and Maritime Engineering: Decommissioning of Offshore Geotechnical Structures, Hamburg, Germany. Keynote 3–40 ISBN-13: 978–3–936310–40–5
7. Gourvenec S, Feng X, Randolph MF, White DJ (2017) A toolbox approach for optimizing geotechnical design of subsea foundations – special session Proc. Offshore Technology Conference, Houston, OTC-27703-MS
8. Guevara M, Doherty JP, Watson P and White DJ (2021) Evolving soil-conductor stiffness due to multiple-episode cyclic loading. Proc. 2nd Vietnam Symposium on Advances in Offshore Engineering (VSOE)
9. Herduin M. (2019) Multidirectional loading characterisation on a shared suction anchor for Wave Energy Converters, PhD Thesis, University of Western Australia
10. Schiavon JA, Tsucha CHC, Thorel L (2017) Cyclic and post-cyclic monotonic response of a single-helix anchor in sand. Géotechnique Letters 7:11–17. https://doi.org/10.1608/grete.16.00100
11. Grievs M, Vickers J (2017) Digital twin: mitigating unpredictable, undesirable emergent behavior in complex systems. In: Kahnlen FJ, Flumerfelt S, Alves A (eds) Transdisciplinary perspectives on complex systems. Springer, Cham, pp 85–113
12. Renzi D, Maniar D, McNeill S and Del Vecchio C (2017) Developing a digital twin for floating production systems integrity management. Proc. Offshore Technology Conference (OTC Brasil), Paper No. OTC-28012-MS, https://doi.org/10.4043/28012-MS
13. Sharma P, Hamedifar H, Brown A, and Green R (2017) The dawn of the new age of the industrial internet and how it can radically transform the offshore oil and gas industry. Proc. Offshore Technology Conference, Paper No. OTC-27638-MS, https://doi.org/10.4043/27638-MS
14. Palmer M, Howard T, Tinker J, Lowe J, Bricheno L, Calvert D, Edwards T, Gregory J, Harris G, Krijnen J. and Pickering M (2018) UK Climate Projections Marine Report UKCP18, available online https://www.metoffice.gov.uk/pub/data/weather/uk/ukcp18/science-reports/UKCP18-Marine-report.pdf
15. Brown A., Stephens A., Rabb B., Connell R. and Upton, J. (2019) Including the Impact of Climate Change in Offshore and Onshore Metocean Design Criteria to Ensure Asset Robustness. Proc. 38th Intnl Conf. Ocean. Offshore & Arctic Engineering, Glasgow, UK ASME OMAE2019–95205
16. Terzaghi K (1943) Theoretical soil mechanics Wiley, New York
17. Andersen KH, Brown SF, Foss I, Pool JH, Rosenbrand WF (1980) Cyclic and static laboratory tests on Drammen clay. J. Geotech. Engng, ASCE 106:499–529
18. Schofield AN, Wroth CP (1968) Critical state soil mechanics. McGraw-Hill, London
19. Roscoe KH, Schofield AN, Wroth CP (1958) On the yielding of soils. Géotechnique 8(1):22–52
20. Sheil B (2017) Numerical simulations of the reuse of piled raft foundations in clay. Acta Geotech 12:1047–1059. https://doi.org/10.1007/s11440-017-0522-8
21. Karlstrud K (2012) Prediction of load-displacement behaviour and capacity of axially loaded piles in clay based on analyses and interpretation of pile load test results. Ph.D. thesis, Ph.D. Thesis, Norwegian University of Science and Technology, Trondheim, Norway
22. Cocjin M, Gourvenec S, White DJ, Randolph MF (2014) Tolerably mobile subsea foundations – Observations of performance. Géotechnique 64(11):895–909. https://doi.org/10.1680/geot.14.P.098
23. Cocjin M, Gourvenec S, White DJ, Randolph MF (2017) Theoretical framework for predicting the response of tolerably mobile subsea installations. Géotechnique 67(7):608–620. https://doi.org/10.1680/geot.16.P.137
24. Hodder MS, White DJ, Cassidy MJ (2013) An effective stress framework for the variation in penetration resistance due to episodes of remoulding and reconsolidation. Géotechnique 63(1):30–43
25. Vulpe C, White DJ (2014) Effect of prior loading cycles on vertical bearing capacity of clay. Int J Phys Model Geotechnics 14(4):88–98
26. Laham N, Kwa KA, White DW, Gourvenec S (2021) Episodic direct simple shear tests to measure changing strength for whole-life geotechnical design. Géotechnique Letters 11(1):1–24. https://doi.org/10.1680/jgeot.20.00124
27. White DJ, Chen J, Gourvenec S and O’Loughlin CD (2019) On the selection of an appropriate consolidation coefficient for offshore geotechnical design Proc. 38th Int. Conf. Ocean and Arctic Engineering (OMAE), Glasgow, UK.
28. Feng X, Gourvenec S (2016) Modelling sliding resistance of tolerably mobile subsea mudmats. Géotechnique 66(6):490–499. https://doi.org/10.1680/geot.15.P.178
29. Boukpeti N, White DJ (2017) Interface shear box tests for assessing axial pipe-soil resistance. Géotechnique 67(1):18–30. https://doi.org/10.1680/geot.15.P.112
30. Kolymbas D (1991) An outline of hypoplasticity. Arch Appl Mech 61:143–151
31. Mašín D (2005) A hypoplastic constitutive model for clays. Int J Numer Anal Meth Geomech 29:311–336
resistance, illustrated by a range of marine clay datasets. Proc. 7th Int. Conf. on Offshore Site Investigation and Geotechnics (OSIG), London, UK. 367–377

69. Yasuhara K, Hirao K and Hyde AFL (1992) Effects of cyclic loading on undrained strength and compressibility of clay. Soils and Foundations, 32(1): 100–116, Japanese Society of Soil Mechanics and Foundation Engineering

70. Yasuhara K (1994) Post-cyclic undrained strength for cohesive soils. J Geotechnical Eng, ASCE 210(11):1961–1979

71. Laham N, Kwa KA, Gourvenec S, White DJ, Deeks AD, & Suzuki Y (2022) Characterizing whole life effect of cyclic loading with intervening consolidation through cyclic DSS testing and modified S-N curves

72. Yasuhara K, Andersen KH (1991) Recompression of normally consolidated clay after cyclic loading. Soils Found 31(1):83–94

Publisher’s Note Springer Nature remains neutral with regard to jurisdictional claims in published maps and institutional affiliations.