

Predicting the pre-peak load–displacement curve of plate anchors in sand: scoping review

28 October 2026. A scoping literature review for a final-year capstone (University of Melbourne). Searching, screening and reading were AI-assisted; every number is taken from the page cited, and figures read by eye are marked as such.

Answer and framing

1. Answer first

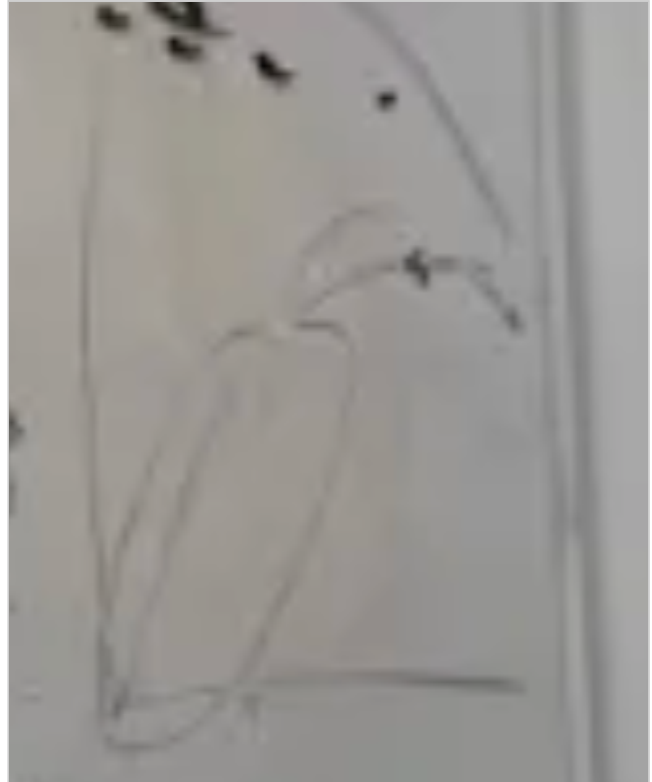
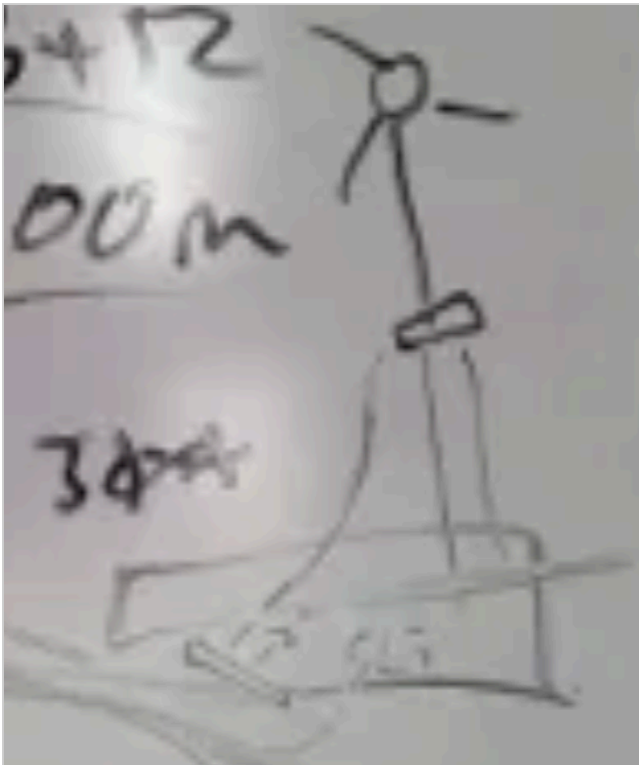
For what the team is predicting, the nonlinear pre-peak part of the load–displacement curve of a horizontal plate anchor pulled out of sand, no paper read for this review fits an equation. The one fitted load–displacement equation read for plate anchors in sand (Ghaly 1997) is for vertical plates pulled sideways and rests on 12 small laboratory tests, which Das and Seeley (1975) had earlier fitted with a hyperbola (reported by Ghaly, p. 75; not opened). The onshore literature supplies a candidate form, the numbers to test it against and a clear gap.

- **What is known.** Twelve plate anchor papers read in full give 87 digitisable curves (55 horizontal plates under vertical pull). Displacement to peak rises with embedment: δ/B about 0.01 to 0.07 at H/B 1 to 3 in dense sand, about 0.12 to 0.2 in loose sand, and 0.2 to 0.3 at H/B 7 to 8. At shallow depth denser sand peaks at less displacement; the order reverses deep. Small 1 g models reach peak at about one-third of the displacement of 1 m centrifuge prototypes (Dickin 1988).
- **Best candidate form.** The hyperbola $Q/Q_u = \delta/(a + b\delta)$. Onshore it was fitted to 75 field uplift tests of spread foundations (Trautmann and Kulhawy 1988) and 158 uplift tests (Phoon et al. 2007). Normalised by depth, the onshore failure displacements (0.5 to 2 % of depth) match the dense-sand anchor tests. On those data the initial stiffness is only 3 to 4 times the secant stiffness at failure, a one-number test for each of the team's curves.
- **The right onshore analogue** is uplift foundations and buried pipes (same breakout mechanism), with pile serviceability work as the statistical template, not piles as the physics.
- **Design codes.** No document opened sets an anchor displacement limit: DNV editions 2002 to 2013, the first floating wind standard DNVGL-ST-0119 (July 2018), API RP 2SK (2005), Indonesia's oil and gas standards decree and the draft SNI 8460. DNVGL-ST-0119 names a serviceability check for grouted rock anchors only, and its plate anchor clauses assume clay. The current DNV-ST-0119 (2025) was not readable.
- **Offshore conditions.** Most published curves are slow, one-way, upright pulls on plates placed in dry sand at small scale. The pull angle at the plate, drainage, stress level and where displacement was measured each change the curve enough to need a column in the dataset or an exclusion rule; repeated loading, installation and load direction on shared anchors are limits to state (section 9).
- **Gap.** The onshore curve is limited by its authors to embedment ratios of three or less; offshore plates sit deeper. Nobody read here builds an anchor curve from an elastic start (Rowe and Booker), a capacity and a data-fitted shape, and nobody treats test type (1 g against centrifuge) as a variable in a pooled anchor dataset.
- **First steps.** Normalise each curve two ways (δ/B and δ/H); draw Chin plots (δ/Q against δ) and compute $K_0 \delta_u/Q_u$; fit the hyperbola and one shape-exponent variant; validate by leaving whole studies out; try symbolic regression only against that baseline.

2. What the project actually is

The object is the curve, not the anchor. The team is predicting the nonlinear, pre-peak part of a plate anchor's load–displacement curve (they call it a stress–strain curve), which governs the serviceability limit state (SLS), to pair with the capacity (ULS) that existing methods already predict. They work through stiffness, the slope of load against displacement, which starts high and falls as the anchor moves. The application is mooring anchors for floating offshore wind: fixed turbines stand on monopiles, and a floating turbine is held by mooring lines tied to anchors in the seabed. Scope they set: sand, dilation included, monotonic loading, published experimental curves only.

Their two sketches show both halves: a floating turbine whose mooring line runs down to an anchor in the seabed, and a curve that rises steeply from the origin and then bends over, the nonlinear part they want to predict.



Their "stress–strain" wording is more apt than it sounds. The degrading-stiffness description of an anchor curve is the same mathematics as the modulus-degradation curve of the soil itself (shear modulus falling from its small-strain value as strain grows). One established design idea, mobilisable strength design, works by scaling one onto the other (section 5).

3. The curve, derived

Everything in this section is my derivation, not taken from a source.

Notation. Q is the load along the line of pull, δ the plate's displacement along that line, Q_u the capacity (peak load), B the plate width or diameter, H the depth to the plate, γ' the effective unit weight of the sand. Tangent stiffness $K = dQ/d\delta$; secant stiffness $K_s = Q/\delta$; initial stiffness $K_0 = K$ at $\delta = 0$.

3.1 How many variables control the curve. The load depends on δ , B , H , γ' , the sand's small-strain shear modulus G_0 , its critical-state friction angle ϕ'_{cv} , its relative density D_r (which sets dilation), particle size d_{50} , plate shape and load inclination β . Taking B and γ' as the repeating variables gives nine dimensionless groups:

$$Q/Q_u = F(\delta/B; H/B, D_r, \phi'_{cv}, G_0/(\gamma'H), B/d_{50}, \text{shape}, \beta)$$

$G_0/(\gamma'H)$ is a rigidity index (stiffness against stress level) and B/d_{50} a grain-scale ratio. Both differ by one to two orders of magnitude between a 1 g model and a field anchor (section 3.4), so a pooled dataset varies in groups that its authors rarely report. With tens of curves, F cannot be fitted freely over eight groups: the equation needs a fixed shape with a few parameters, and those parameters are then made functions of the groups.

3.2 The hyperbola, written as stiffness. The usual first candidate is $Q = \delta/(a + b\delta)$. Then

- initial stiffness $K_0 = 1/a$, asymptote $Q_\infty = 1/b$, reference displacement $\delta_r = Q_\infty/K_0 = a/b$;
- secant stiffness $K_s = 1/(a + b\delta) = K_0 (1 - Q/Q_\infty)$;
- tangent stiffness $K = a/(a + b\delta)^2 = K_0 (1 - Q/Q_\infty)^2$.

So the hyperbola is a statement that secant stiffness falls linearly with load ratio. Two plots test it on the team's existing stiffness data with no fitting: δ/Q against δ should be a straight line (slope b , intercept a), and K_s/K_0 against Q/Q_∞ should be a straight line from 1 to 0. Curvature in either plot is evidence against the hyperbola and tells you which way to bend it.

In normalised form $Q/Q_\infty = x/(1 + x)$ with $x = \delta/\delta_r$. The load reaches 50 % of the asymptote at $x = 1$, 75 % at $x = 3$, 90 % at $x = 9$ and 95 % at $x = 19$. A hyperbola therefore never peaks: if the measured peak Q_p occurs at δ_p , setting $Q_\infty = Q_p$ forces the fitted curve below the data near the peak, and dense sand, which peaks and then softens, cannot be represented at all.

3.3 Capacity and initial stiffness do not fix the middle. Onshore uplift practice does not put the failure load at the hyperbola's asymptote. Trautmann and Kulhawy (1988) pin their curve so that $Q/Q_u = 1$ at $z/D = 0.04$, which puts the asymptote at $Q_u/0.67 = 1.49 Q_u$; Phoon et al. (2007) find a mean b of 0.75 for spread foundations, an asymptote of $1.33 Q_u$ (section 5). Writing the normalised hyperbola as $Q/Q_u = \delta/(a + b\delta)$, the load reaches Q_u at $\delta_u = a/(1 - b)$, so

$$K_0 \delta_u / Q_u = 1/(1 - b) = 3.0 \text{ (} b = 0.67 \text{) to } 4.0 \text{ (} b = 0.75 \text{):}$$

on the onshore uplift data, the initial stiffness is only three to four times the secant stiffness at failure. That ratio is a one-number test per curve the team can compute from their stiffness data. Two consequences. First, the "initial" stiffness of these fits is the slope at the first measured load step, already degraded; an elastic K_0 computed from the small-strain modulus G_0 (section 5, Rowe and Booker) will be stiffer, and a hyperbola built on it will rise too fast unless E is taken at a working strain or a shape exponent is added. Second, if the team's anchor curves give $K_0 \delta_u/Q_u$ well outside 3 to 4, the onshore hyperbola does not transfer as is. The two ends (K_0 , Q_u) can come from existing methods; the shape between them is what the data must supply, and it is the project's real contribution. A three-parameter degradation form, written directly in the team's stiffness language, is

$$K_s/K_0 = 1/(1 + x^\alpha), \text{ that is } Q = K_0 \delta/(1 + x^\alpha), \text{ with } x = \delta/\delta_r.$$

$\alpha = 1$ recovers the hyperbola. For $\alpha > 1$ the curve has a peak: $dQ/d\delta = 0$ where $x^\alpha = 1/(\alpha - 1)$, so $\delta_p = \delta_r (\alpha - 1)^{-1/\alpha}$ and $Q_p/(K_0 \delta_p) = (\alpha - 1)/\alpha$. Example: if the secant stiffness at peak is 5 % of K_0 , $(\alpha - 1)/\alpha = 0.05$, so $\alpha = 1.053$ and $\delta_p = 19^{0.95} \delta_r \approx 16.4 \delta_r$. This form is constructed here as a candidate with the same shape as the modified hyperbolic modulus-degradation curves used for soils; it is untested on anchor data.

3.4 Stress level and scale, in numbers. A 1 g model plate 50 mm wide at $H/B = 3$ sits under $16 \text{ kN/m}^3 \times 0.15 \text{ m} = 2.4 \text{ kPa}$ of vertical effective stress. A 3 m field plate at $H/B = 3$ in sand with $\gamma' = 10 \text{ kN/m}^3$ sits under $10 \times 9 = 90 \text{ kPa}$, 37.5 times more. Grain scale: B/d_{50} is $50/0.3 \approx 170$ for the model in a 0.3 mm sand, and $3000/0.3 = 10,000$ in the field. Sand dilates more at low stress, so 1 g curves are expected to show more dilation-driven strength and a different shape than the field. Spinning the same 50 mm model at 37.5 g raises its stress to 90 kPa, matching the field, but B/d_{50} stays at about 170. The measured effect is large: 50 mm anchors at 1 g in dense sand reached peak at "only one-third" of the displacement of 1 m

prototypes modelled in the centrifuge (Dickin 1988, p. 1308), and their breakout factors would give "overoptimistic predictions" if extrapolated (p. 1307). Section 4 gives the rest.

3.5 How many independent data the team has. Points along one curve are not independent: each comes from the same plate, sand, rig and operator. Curves from one paper share all but one or two of those. If 40 curves come from 8 papers, an equation's ability to predict a new site is tested on 8 cases, not 40 and not 2,000 digitised points. Leave-one-study-out validation (fit on 7 papers, predict the 8th, rotate) measures exactly that. A random split of points measures interpolation within curves the model has already seen and will look far better than it is.

Evidence

4. Data available

Twelve papers read in full give 87 load–displacement curves that can be digitised, 55 of them horizontal plates pulled vertically, plus about 11 finite element curves; none fits an equation to the pre-peak part.

Source	Test	Sand	Plate	H/B	Curves	δ at peak / B
Ilamparuthi et al. 2002	1 g	Palar River sand, I_D 39, 62, 74 %	Circular, 100 to 400 mm	to 12	4 (Fig. 6), plus 40 tabulated peaks (Table 2)	Dense: 0.013 at H/D 1.9 to 0.25 at 12
Dickin 1988	Centrifuge 40 g (1 m prototype); 1 g (50 mm)	Erith sand, RD 76 and 33 %	Square and rectangular	to 8	3 (Fig. 5), δ_f/B against H/B (Fig. 10)	Dense $\approx$ 0.03 at H/B 1 to $\approx$ 0.30 at 8; loose $\approx$ 0.20 at H/B 3 (by eye)
Murray and Geddes 1987	1 g	D_r 86 and 42 %	Rectangular 50.8 mm; circular 50.8 to 88.9 mm	to 10	3 (Fig. 2), failure-displacement points	Very dense $\approx$ 0.01 at H/B $\leq$ 4 to $\approx$ 0.05 at 10 (by eye)
Sakai and Tanaka 1998	1 g	Toyoura sand, D_r 0.99	Circular, 3 to 20 cm	1 to 3	9 (Figs 4 to 6)	0.012, 0.025, 0.035 at h/D 1, 2, 3, any diameter
Giampa et al. 2019	1 g	Westerly sand, I_D $\approx$ 0.23 and 0.55	Circle, square, triangle, kite, 127 to 305 mm	1 to 5	15 (Fig. 4, loose)	$\approx$ 0.2 at H/B 1 to 3, $\approx$ 0.5 deeper
Bradshaw et al. 2016	1 g	Westerly sand, I_D $\approx$ 0.20	Square 152 and 305 mm	1 to 5	3 (Fig. 7)	$\approx$ 0.05 at H/B 1 to $\approx$ 0.3 at 5 (by eye)
Zhuang et al. 2022	1 g, plane strain	Silica sand, D_r 50 and 83 %	Strip, 50 mm	1 to 7	48 (Fig. 3; 16 horizontal), δ/H axis	0.95 to 12.9 % of H
Liu et al. 2012	1 g, half model	Fine and coarse, loose and dense	Circular 50.8 mm	1 to 9	Fig. 3 (not counted)	Loose fine sand, H/D 3: 0.12
Kurniadi et al. 2026	Centrifuge 30 g, saturated, drained	UWA silica sand, I_D 85 %	Circular, 0.9 m prototype	3	2 monotonic (Fig. 5)	0.07
Al Hakeem and Aubeny 2019	Finite element	ϕ' 30 to 50°, ψ 0 to 25°, rigidity index 100 to 500	Circular	8 to 20	8 (Fig. 9)	Rigidity index 500: $\approx$ 0.28 at z/D 8

Capacity-only data also exist: [Roy et al. 2021](#) report 60 centrifuge tests and compile 161 earlier tests: useful for Q_u , not for the curve. Displacements marked "by eye" were read from rendered figures, not digitised; treat them as $\pm$ 0.02 to 0.05 in δ/B . Zhuang et al. and Roy et al. were read as submitted manuscripts; Giampa et al. and Bradshaw et al. as typeset copies whose version the cache could not confirm.

Displacement to peak rises with embedment in every source, and in dense sand it agrees with the onshore analogues. Sakai and Tanaka (1998, p. 95) give s/D 0.012, 0.025 and 0.035 at h/D 1, 2 and 3; Ilamparuthi et al. (2002, Table 2, p. 654) 0.013 at H/D 1.9 rising to 0.25 at 12; Dickin (1988, Fig. 10) about 0.03 at H/B 1 to 0.30 at 8. At $H/B = 3$ the dense values (0.035 to about 0.06, and 0.07 in Kurniadi et al.'s centrifuge test, p. 04025170-5) are 1.2 to 2.3 % of depth, the same order as the 0.5 to 2 % of depth at which onshore spread foundations and pipes mobilise (section 5).

At shallow depth denser sand peaks at less displacement; deeper than about H/B 6 the order reverses. Murray and Geddes (1987, p. 206) report "considerably less" displacement in very dense sand; Liu et al. 2012 find it "higher in loose sand", with "a similar trend" in other studies (p. 732); Dickin's Fig. 10 shows about 0.20 loose against 0.06 dense at H/B 3. Loose-sand displacement levels off near H/B 6 while dense keeps rising (Dickin, pp. 1307 to 1308), which matches Ilamparuthi et al. (p. 654: dense larger beyond H/D 6). Density therefore has to enter the equation through an interaction with depth, not as a separate multiplier.

Small 1 g models understate displacement to peak; whether plate size alone matters at 1 g is unresolved. Dickin's one-third result (section 3.4) compares 1 g with centrifuge, that is, stress level. Sakai and Tanaka, all at 1 g, found the breakout factor fell as diameter grew from 3 to 20 cm while s/D at peak stayed constant at a given h/D (p. 95), and attribute the scale effect to particle size and progressive failure (p. 93). Ilamparuthi et al.'s Table 2 points the other way (δ/D 0.013 for a 100 mm plate against 0.027 for 400 mm at $H/D \approx 2$). So test type must be a variable in the pooled dataset, and plate size at 1 g should be tested, not assumed away.

What the data cannot do yet. Few measured curves go deeper than H/B 6. The best deep series is a centrifuge set on 0.4 m plates in dense sand to H/D 12 (Hao et al. 2019, tabulated in Kurniadi's 2024 University of Melbourne PhD thesis), and beyond H/D 6 those curves level off rather than peak (section 9.5). Offshore plates are often keyed or dynamically installed; no curve read compares an installed plate with a pre-placed one (section 9.7), and Chow et al.'s dynamically installed plate study (Ocean Engineering) would not open. Shapes, sands, rates and drainage vary between papers. Of the 87 curves, 48 come from one plane-strain study (Zhuang et al.) and 32 of those are inclined plates, so the dataset rests on eight independent studies, not 87 independent curves (section 3.5).

5. Equation forms

Two fitted forms were read: a hyperbola fitted to uplift tests on shallow spread foundations and piles, and a power law fitted to 12 laboratory tests on vertical plate anchors pulled sideways (Ghaly 1997), whose source, Das and Seeley (1975), fitted the same tests with a hyperbola (not opened). No paper opened fits an equation to the pre-peak curve of a horizontal plate pulled upward. A later thesis describes two 1981 papers as proposing "a hyperbolic equation to describe the anchor response under cyclic loadings" (Schiavon 2017, pp. 50 to 51, citing Andreadis and Harvey 1981; see also Andreadis et al. 1981). Neither could be opened, so they are probable exceptions whose equation, soil and data are unknown.

Form	Equation	Fitted to	Reported fit	Source
Normalised uplift hyperbola	$Q/Q_u = (z/D)/(0.013 + 0.67 z/D)$; z upward displacement, D depth to base	75 full-scale field uplift tests, 17 sites, spread foundations (grillage, slab, plate) in granular and cohesive backfill, $D/B \leq 3$	Not a regression: pinned at $z/D = 0.01$ at $Q_u/2$ and $z/D = 0.04$ at Q_u (95 % upper limit). Geometric mean z/D at failure 0.47 % (granular slab) to 1.83 % (cohesive slab)	Trautmann and Kulhawy 1988, pp. 177, 180

Form	Equation	Fitted to	Reported fit	Source
Probabilistic hyperbola	$Q/Q_u = y/(a + by)$; 1/a initial slope, 1/b asymptote	158 uplift tests: 85 spread foundations, 48 drilled shafts, 25 pressure-injected footings; sand and clay pooled	Spread: a 7.13 mm (COV 0.65), b 0.75 (COV 0.18); a-b correlation -0.24 (spread), -0.59 (shafts), -0.73 (pressure-injected)	Phoon et al. 2007 , pp. 3, 6, 7
Hyperbola, helical and expanded piles	$F = \delta/(a\delta + b)$; 1/a limit, 1/b initial stiffness	80 full-scale tests (37 tension), 27 sites, mixed soils	No curve statistic; measured maximum over hyperbolic limit 0.68 to 1.38	Rahimi et al. 2024 , accepted manuscript
Hyperbola on slope-tangent capacity	$Q/Q_{STC} = \delta/(k_1 + k_2 \delta)$	95 augered cast-in-place piles, granular soil, compression	k_1-k_2 Kendall tau -0.72	Reddy and Stuedlein 2017 , accepted manuscript
Pipe uplift hyperbola	$F'' = Z''/(0.93 + 0.07 Z'')$ as printed; F'' force over peak, Z'' displacement over displacement at peak	13 model pipe tests, loose to dense sand, H/D up to 13	Graphical only. Probable misprint: the swapped form $Z''/(0.07 + 0.93 Z'')$ reproduces the paper's own bilinear point and $K_{v70} = 500 N_v \gamma D L$; check Fig. 9 before use	Trautmann et al. 1985 , pp. 1071 to 1072
Power law, vertical plates pulled sideways	$P/P_u = 2.2 (u/H)^{0.3}$; P horizontal load, u horizontal displacement, H depth; gives $u = 0.072H$ at P_u	12 laboratory 1 g tests (Das and Seeley 1975): 51 mm plates, b/h 1, 3 and 5, ϕ 34°, H/h 1 to 4. The paper's 128-test database (104 laboratory, 15 centrifuge, 9 field) is used for capacity only	No fit statistic for this equation; infinite initial stiffness and no density term; the published discussion and erratum do not change it	Ghaly 1997 , pp. 75 to 76
Trilinear pipe uplift (code)	Breaks at $\delta/\delta_f = 0.2$ ($R/R_{max} = \alpha$, mean 0.78) and $\delta_f = 0.005$ to 0.010 H	Trautmann et al. 1985 data	COV of α 0.10	DNV-RP-F110 (October 2007), App. B.3.3
Power law (clay)	$\tau_{mob}/c_u = 0.49 (\gamma/\gamma_{M=2})^{0.6}$	115 curves, 19 clays and silts	R^2 0.90, n 1365	Vardanega and Bolton 2011 , p. 1489
Mobilisable strength design (clay)	$\epsilon_{mob} = M_c \delta/D$ with $M_c = 1.35$; $\sigma_{mob} = N_c c_{mob}$	Undrained circular footings on clay	Not checked	Osman and Bolton 2005 , p. 437
Elastic initial stiffness	$P/(B \delta E) = I_D I_h/(1 - \nu^2)$	Elastic analysis, rigid circular and strip anchors	Matches Selvadurai's deep bonded plate to better than 1 % (p. 201)	Rowe and Booker 1979 , p. 200

Displacement to mobilise capacity is consistent across the onshore analogues: about 0.5 to 2 % of embedment depth in dense or compacted ground, and up to about 5 % in loose sand. Spread foundations fail at geometric-mean displacements of 0.47 to 1.83 % of depth (Trautmann and Kulhawy 1988, Table 3, p. 177). Pipes in sand peak mostly at 0.5 to 1.5 % of depth, with 0.01H recommended for design ([Trautmann et al. 1985](#), pp. 1069, 1071). Reviewing centrifuge pipe tests, [Cheuk et al. 2008](#) put the peak at about 1 to 2 % of depth in dense sand and up to about 5 % in loose sand (p. 155); their own four tests peaked at 1 % of depth (p. 160). At H/B = 3 those fractions are δ/B of about 1.5 to 6 % in dense sand, which is the range the plate anchor tests show (section 4). Cheuk et al. conclude that both δ/D and δ/H matter, which is a direct warning for the team's normalisation: dividing by plate width alone will make deep and shallow curves look different when part of the difference is just depth.

Mobilisable strength design is the onshore idea closest to the team's "stress-strain curve" reading, and it has not been applied to plate anchors in sand in anything found here. [Osman and Bolton 2005](#) take "the non-linearity of the representative stress-strain curve ... as identical to that of the normalised

load–displacement curve of the foundation", with two mechanism constants converting strain to displacement ($M_c = 1.35$) and stress to pressure (N_c) (p. 437). The anchor's stiffness degradation then is the soil's, stretched on both axes. The method is for undrained clay. A sand version would need the uplift mechanism geometry (White et al. 2008, "The uplift resistance of pipes and plate anchors buried in sand", *Géotechnique*, not opened: Emerald gave the abstract page only) and a sand modulus-degradation curve (Oztoprak and Bolton 2013, not opened). In the 40 top OpenAlex hits for "mobilizable strength design" (66 in total, 28 Oct 2026) I saw no anchor or uplift application; that is a title screen, not a search for absence.

An elastic solution can fix the start of the curve, but no opened paper has used it that way. Rowe and Booker 1979 give $K_0 = E B I_D I_h / (1 - \nu^2)$ for a rigid circular plate (p. 200). Read by eye from their charts, $I_D \approx 2.2$ for a deep soil layer at $\nu = 0.3$ and I_h rises from about 0.5 at the surface to about 0.9 at $h/B = 3$ and 1 at depth, so $K_0 \approx 2.2 E B$ at $h/B = 3$. Their basic case assumes the soil stays bonded under the plate; immediate breakaway lowers the stiffness (p. 199), and a plate pulled upwards does separate underneath. Section 3.3 explains why E must be taken at a working strain rather than from G_0 if this K_0 is to start a hyperbola fitted to load-test data.

6. Piles or anchors as analogue?

Steelman for piles. Piles have the largest load-test databases and the most developed serviceability frameworks: hyperbolic curves with correlated, probabilistic parameters (Phoon et al. 2007; Reddy and Stuedlein 2017), fitted to tension tests as well as compression (Rahimi et al. 2024, 37 tension tests). Helical piles are multi-plate anchors on a shaft. A team that knows pile design already knows the curve-fitting machinery.

Devil's advocate. A pile's response is dominated by shaft friction along a long interface; a plate anchor's by the breakout of a soil mass above the plate, which is why its displacement to failure scales with depth (section 5). Trautmann and Kulhawy, the closest onshore data, say their curve "should not be used for deep plates, anchors, or other foundation elements where the embedment ratio is greater than three" (p. 180) and may be unconservative in loose backfill (p. 182). Pooled pile parameters also mix sand and clay (Phoon et al. 2007, p. 6).

Verdict. Use onshore uplift foundations (transmission tower spread foundations, buried pipes) as the physical analogue: same mechanism, same normalisation by depth, numbers that agree with the anchor tests. Use the pile serviceability literature as the statistical template: two-parameter curves whose parameters are treated as a correlated random pair. The team's premise, "onshore pile behaviour is predictable", survives only in that second sense. Deep plates (H/B above about 3) have no onshore load-test analogue among the papers opened here; they rely on numerical studies and the elastic solution, which is a gap (section 10).

7. Design guidance on anchor displacement

No guidance opened for this review sets a displacement, stiffness or serviceability limit for an anchor in soil, and none covers plate anchors in sand; the current editions could not be opened, so that statement has an edition boundary. Anchors are designed for capacity in the ultimate and accidental limit states. Details:

- DNV-RP-E302 (December 2002, plate anchors in clay) uses displacement only as an installation allowance: App. I4 "tentatively" takes the failure displacement as 30 % of plate width, $\Delta z_f = (0.3 \pm 0.1) W_F$, added to the target depth. Searches for "serviceab", "SLS", "stiffness" and "load-displacement" in it returned nothing.
- DNV-OS-E301 (October 2013, position mooring), Ch.2 Sec.4 [3.5.4]: the only displacement check is that the drag of a fluke anchor in a 100-year storm is "tolerable", case by case.

- DNV-OS-J103 (June 2013, floating wind turbines, the forerunner of DNV-ST-0119), Sec.9 [1.1.5]: anchor resistance is analysed for ULS and ALS; an SLS check is named only for grouted rock anchors.
- DNVGL-ST-0119 (July 2018, floating wind turbine structures; the first edition of the current floating wind standard, read from a copy of the 2018 edition): unchanged from OS-J103. Clause 9.1.1.5 (p. 112): "The analysis of anchor resistance shall be carried out for the ULS and the ALS ... For grouted rock anchors, analysis of the anchor resistance may also be necessary for the SLS." Section 9.7 (p. 122) covers plate anchors in clay only: "The anchor resistance is a function of the undrained shear strength at the penetration depth in question, of the plate area and of the effect of cyclic loading" (9.7.1.3), design by DNVGL-RP-E302, soil material factor at least 1.4 in the ULS (9.7.2). It defines the SLS generally as "deformations in excess of tolerance without exceeding the load-carrying capacity" (Sec. 1, p. 25) with load factor 1.0 (5.1.3, p. 77), but sets no tolerance for an anchor in soil.
- API RP 2SK (3rd edition, October 2005; a scanned copy, read by OCR of section 7 and Appendix E; the 2024 4th edition was not available): App. E says that for anchors of deepwater floaters "the central focus is the ultimate capacity of the suction anchor and not the load deflection behavior" (p. 98), and that "for the purpose of suction anchor design, only the ultimate limit state is considered" (structural design, p. 111). Its safety factors for pile, plate and gravity anchors (Table 7, section 7.4.2) are on capacity.
- DNV-RP-F110 (October 2007, pipeline buckling), App. B.3.3: the one open document with a normalised pre-peak curve in sand. Uplift resistance of a buried pipe is "fully mobilized at a vertical uplift displacement δ_f , where δ_f is 0.005-0.010 times the height H", on a trilinear curve with the first break at $\delta/\delta_f = 0.2$ and $R/R_{max} = \alpha$, $E[\alpha] = 0.78$, COV 0.10, calibrated on Trautmann et al. (1985).
- Indonesia: the oil and gas standards decree, Kepmen ESDM 1846 K/18/MEM/2018 (now revoked by 283.K/MG.01/MEM.M/2025, whose list could not be opened), sets no anchor criterion of its own and names ISO 19901-4, ISO 19901-7, API RP 2SK, API RP 2GEO, API RP 2T and DNV-OS-C101 for deepwater and subsea foundations (Lampiran B item 1). The April 2017 draft of SNI 8460:2017 tests ground anchors on elastic extension (between that of 90 % and 110 % of the free length, 10.6.5.1) and load loss (1 % at 5 min, 2 % at 15 min, 10.6.5.2, from BS 8081), and limits tension pile movement at 200 % of design load to PL/EA + 4 mm, at most 25 mm (9.8.2). These are proof-test acceptance rules, not serviceability limits. No SNI or SPLN on transmission tower foundations was found. The published SNI 8460 and the BKI class rules would not open.

Why it matters even without a clause: mooring analyses treat the anchor as a fixed point, "neglecting potential loss of pretension due to anchor displacement" ([Cerfontaine et al. 2023](#), p. 4). In a taut mooring the line is stiff along its length, so anchor movement along the line changes pretension and platform offset directly (my reasoning, not a quoted finding). That is the engineering case for the project, and the capstone should make it in those terms rather than claim a code requirement. The 2018 edition of the floating wind standard, the 2013 document it replaced and API RP 2SK (2005) all leave anchor displacement unchecked, and the 2018 plate anchor clauses are written for clay. **Before the team writes "no code sets an anchor SLS" as a present-tense fact, the anchor section of DNV-ST-0119 (2025-12) should still be read** (subscription only; no copy could be found on 28 October 2026); until then, write "no edition up to 2018".

For the team

8. Finding the equation

Fit a chosen physical form first and let symbolic regression compete with it second; validate both by leaving whole studies out. The reasons are in section 3.5 and below:

- *Regression on a chosen form* (hyperbola, power law, modified hyperbola of section 3.3, the DNV-RP-F110 trilinear as baseline) has two or three parameters per curve, which a few dozen curves can

identify. Its parameters can then be regressed on H/B, D_r and test type, ideally in a mixed-effects model with study as a random effect, so between-study differences are estimated rather than hidden.

- *Symbolic regression* (PySR, [Cranmer 2023](#)) returns a Pareto front of accuracy against expression size. On few independent curves many expressions fit equally well, so complexity must be chosen by held-out studies, repeated over seeds, and checked for zero load at zero displacement and a monotonic rise. Run it over the dimensionless groups of section 3.1, not raw variables.
- *AI Feynman* ([Udrescu and Tegmark 2020](#)) generated 100,000 points per equation (p. 4); its useful part here is the dimensional analysis, which section 3.1 already does by hand.
- *Sparse regression* (SINDy, [Brunton et al. 2016](#)) is built for time histories and their derivatives; applied to stiffness from digitised curves, differentiation noise dominates.
- *Black boxes* (boosted trees, polynomial chaos) are only an accuracy ceiling. The closest anchor use found, [Wang et al. 2022](#), predicts one mobilisation distance for helical anchors in dense sand with random folds, which mix points from the same test across training and testing.

Validation protocol: outer loop leave-one-study-out (the test number); every choice of form, complexity or threshold made in an inner loop on the training studies (the validation number); report error per held-out study and in the working range Q/Q_p 0.2 to 0.8, where a good fit near peak can hide a poor fit at service load. An equation that does not beat the F110 trilinear under this protocol has not earned its extra terms.

One pooling trap found while reading: [Chow et al. 2015](#) measured mooring line displacement at the actuator, and the first part of their curve is the line cutting through the soil while the plate keys (accepted manuscript, p. 5). Line displacement and plate displacement must not share a column.

9. Offshore conditions the published curves leave out

Most published curves are slow, one-way, upright pulls on horizontal plates placed in dry sand at small scale, with displacement measured at the plate. A seabed plate under a floating wind mooring differs in seven ways. Four change the curve enough that every pooled curve needs a column for the condition or an exclusion rule: the pull angle at the plate, where displacement was measured, drainage, and stress level with depth. Three are limits to state: repeated loading, installation, and load direction on shared anchors.

How this section was searched: 12 Boolean queries on OpenAlex plus one step of backward and forward citation searching from 11 seed papers gave 1,367 records (28 October 2026). They were ranked by similarity to the seeds and screened on title and abstract in blocks of 250, in rank order; screening stopped when blocks fell below 5 % includes (ranks 751 to 1,250: 3.6 % and 0.4 %), after 147 includes and 526 possible includes. Ranks 1,251 to 1,367 were not screened, and arXiv could not be searched (rate-limited). One relevant paper turned up outside the pool ([Lee et al. 2019](#), cyclic inclined loading of helical anchors), so the search has known misses.

Condition	What it does to the pre-peak curve	Main evidence	What to do with the dataset
Pull angle at the plate	Within about 45° of vertical, little change in normalised shape, but displacement to peak can nearly double in dense sand; by 90°, 2 to 9 times, and the peak can vanish	1 g plate tests; centrifuge and finite element	Record the angle at the plate; pool 0 to 45° from vertical with angle as a column
Where displacement is measured	Line slack and keying move the apparent peak from 0.05 to 0.1 B out to 0.7 to over 3 B	Centrifuge	Exclude line-displacement curves
Drainage and rate	Same first 2 to 3 % of B, then peak load up to about 5 times and displacement to peak 3 to 9 times	Centrifuge; finite element	Fit drained curves only
Stress level and depth	1 g tests understate displacement to peak 2 to 3 times; from H/D 6 there is at most a slight peak	Centrifuge against 1 g	Stress-level column; one rule for "peak"

Condition	What it does to the pre-peak curve	Main evidence	What to do with the dataset
Repeated loading	The one-way curve supplies the first-cycle displacement in the only accumulation law read; drained cycling reached twice the displacement to peak without failure	Centrifuge	Present the equation as the first-loading curve
Installation	Not measured for plates; for a helical anchor, installation halved capacity and softened the curve after a few millimetres	Centrifuge; discrete element	State as a limit
Load direction and level	Shared anchors are mostly loaded within 10° of the weather direction; mean loads are slow, storm loads come at wave periods	Mooring analyses	Report the curve at mean and design load

9.1 Pull angle and plate orientation. Within about 45° of vertical the normalised curve shape changes little, but displacement to peak can already rise 1.75 to 2.1 times in dense sand (1 g plates inclined and pulled normal to their face); by 90° it is 2 to 9 times and the peak can disappear. For the team: record the pull angle at the plate; pool 0 to 45° from vertical as one population, and treat vertical plates pulled sideways as a separate case, which is where Ghaly's power law (section 5) sits. [Hanna et al. 2014](#) (1 g, 152.5 mm square plate pulled along its normal, D_r 63 %) found displacement to peak rising from δ/B 0.105 for a vertical pull to 0.246 for a horizontal one (Fig. 4, p. 113, read by eye), while their curves normalised by peak stayed within about ± 0.07 in Q/Q_p of each other (Fig. 3, p. 112, by eye). [Zhuang et al. 2022](#) (1 g strip plates inclined and pulled along their axis) give displacement at peak within about 10 % of the vertical-pull value up to 45° in medium-dense sand, about twice it by 45° in dense sand at H/h of 3 or more, and 2.4 to 9.3 times it at 90° (Table 2, submitted manuscript); at 90° the curve has no peak (Fig. 3). In finite element analyses of horizontal plates, curves at 45 to 90° from horizontal nearly coincide once normalised ([Cheng et al. 2025](#), Fig. 7a, by eye), and the displacement to the capacity plateau is δ/B 0.01 to 0.02 up to 60° from vertical, rising to about 0.07 at 85° in dense sand, with loose-sand curves still rising at δ/B 0.10 to 0.18 ([Roy et al. 2022](#), Fig. 9, by eye, submitted manuscript; its text states the opposite trend, and the figure is reported here). Capacity rises with angle, by about 1.13 times at 30°, 1.35 at 60° and 1.7 to 1.85 at 90° from vertical (Roy et al., p. 8). The dead-weight tests of [Tilak and Samadhiya 2023](#) have no peak (their "peak" is the load at the end of travel), so they cannot supply a displacement to peak.

9.2 What angle the plate sees. In the two centrifuge series read, a padeye several metres down under a catenary mooring was pulled at about 30° to 90° from horizontal (27.5 to 64.5° in the second), not the 0 to 15° often quoted, and the pull started near vertical and flattened as the load grew. For the team: in the one deep dense-sand test (Lee et al. 2014, Test 1, 10.5 m) the line stayed vertical at about 6 MN because it never cut through the sand, and every test started vertical partly because the wire was installed vertical, so upright pull-out of a horizontal plate is a reasonable analogue for a deep plate in dense sand at service load, not a demonstrated one. State 45 to 90° from horizontal (0 to 45° from vertical) as the range the pooled vertical-pull curves cover, flag that the padeye angles of 30 to 45° from horizontal measured at 5.5 m depth lie outside it, and treat the angle as an output of line tension, padeye depth and sand strength. The quoted ranges, 0 to 15° at the padeye for catenary moorings and 30 to 45° for taut ones ([Cerfontaine et al. 2023](#), p. 4, citing Randolph and Gourvenec 2011), describe a shallow attachment. Below the seabed the line cuts an inverse catenary: with a bearing pressure on the line of $q = N_q \gamma' z$, the angle at the padeye follows, for a line flat at the seabed, from $T_a \theta_a^2/2 = \int Q dz$ (within about 10 % up to 45°), and the friction ratio drops out ([Neubecker and Randolph 1995](#), eqs 4 and 6, pp. 798 to 799). In centrifuge tests with a wire lying flat on the seabed and 5.6 to 6.4 MN of tension, the measured angle at the padeye was 30° (D_r 51 %) and 45° (D_r 76 %) at 5.5 m depth, and 55° and 90° at 10.5 m ([Lee et al. 2014](#), Table 4, p. 7). The line stayed vertical for the first 37 to 50 % of the load (Fig. 6b), partly because it was installed vertical; the paper's figures end 17 to 27° steeper than its table, and the table is reported here. In other centrifuge tests the padeye load also started vertical and rotated to 27.5 to 64.5° from horizontal as load built up ([Kwa et al. 2025](#), accepted manuscript).

9.3 Where displacement is measured. Curves plotted against mooring-line displacement are not plate curves. For the team: exclude any curve whose displacement was measured at the line, padeye or actuator unless the paper gives plate displacement or the line compliance to subtract. In Roy et al.'s (2022) inclined tests the 30, 60 and 90° curves are plotted against line displacement, which includes slack: the apparent peak moves from δ/B 0.05 to 0.10 for vertical pulls to 0.7 to over 3 times B, behind an S-shaped toe that no soil law produces (Fig. 6, by eye; p. 8). Keyed plates are reported the same way: the displacements at peak of 2.65 to 6.27 B in Chow et al. 2020 are mooring-line displacements at the mudline, after a stage in which the line cuts through the soil while the plate rotates (accepted manuscript; corrigendum Table 2).

9.4 Drainage, loading rate and saturation. In saturated sand, faster pull-out leaves the first 2 to 3 % of B of the curve unchanged but multiplies the peak and the displacement to it, so drained and partially drained curves cannot share one normalised shape. For the team: fit drained curves only, record V for any saturated test, and describe the equation as the drained, first-loading curve. In centrifuge tests on 1 × 2 m (prototype) plates at H/B 2.5 to 2.9, every curve followed the same path to δ/B of about 0.02 to 0.03 (Fig. 5, by eye); drained peaks came at δ/B 0.10 (dense) and 0.15 (medium dense), and the undrained (plateau) tests reached 4.6 (dense) and 5.3 (medium dense) times the drained capacity at 3.0 and 4.5 to 5.6 times the displacement (Roy et al. 2022, consolidation study, accepted manuscript, PDF pp. 10 to 12). Finite element analyses give 1.8 to 4.3 times the capacity and 4.0 to 9.3 times the displacement to peak for undrained against drained uplift in dense sand (Liu 2026, PhD thesis, University of Melbourne and University of Toronto, PDF p. 143). Drainage is governed by the normalised velocity $V = vd/c_v$: a horizontal plate is drained only below $V \approx 1$, with d the radius of a circle of equal area (Roy et al., consolidation study, PDF p. 13), against $V \approx 16$ for a keyed plate with V defined differently (Chow et al. 2020). With $c_v = 4.9 \times 10^{-4} \text{ m}^2/\text{s}$, as measured for a fine silica sand (Chow et al. 2020, corrigendum Table 2), a 3 m square plate (equivalent radius 1.7 m) moving 3 to 5 % of B in a quarter of a 12 s wave period gives V of about 100 to 170 (my arithmetic), so wave loads on metre-scale plates in fine sand are likely partially drained, while mean loads are drained. Drained capacity in dry and saturated sand agreed within 0 to 10 % once normalised by $B\gamma'/p_a$ (Roy et al., consolidation study, PDF p. 11); that compares capacity, not curve shape. The thesis warns that the partial-drainage gain "should not be regarded as an unconditional design benefit" (Liu 2026, PDF p. 304).

9.5 Stress level and depth. Small 1 g models understate displacement to peak two to three times, and deep plates show at most a slight peak. For the team: carry stress level ($\gamma'H/p_a$, or plate size with g level) and, where available, the state parameter as columns; record how each source defined the peak and apply one rule to all. For vertical plates pulled sideways, relative failure displacement at H/h 7 was 13 % at 1 g against 42.5 % for a 1 m prototype at 40 g, a factor of 2.3 at H/h 1 rising to 3.3 at H/h 7 (Dickin and Leung 1983, Fig. 25, p. 1520, by eye), in line with the factor of about three for horizontal plates (section 3.4). For pooling small and large tests, matching the sand's state parameter (its void ratio relative to the critical state line at the test stress) reproduced centrifuge results better than matching a dilatancy index, especially at depth (Liu 2026, PDF pp. 263 and 300). With depth, measured displacement to peak in dense sand rises from δ/D 0.05 at H/D 3 to 0.14 at 6, 0.20 at 9 and 0.24 at 12 (Hao et al. 2019 centrifuge tests on 0.4 m plates, as tabulated in Kurniadi 2024, PhD thesis, University of Melbourne, Table 3.2, p. 72). From H/D 6 the curves level off near δ/D 0.2, or soften only slightly, so the "peak" is set by convention, there the point where tangent stiffness falls to 6 % of its initial value (p. 70). In the stiffness form of section 3.3, the hyperbola reaches that point at $Q = 0.755 Q_\infty$ ($x \approx 3.1$). Numerical models are not yet a substitute for measured curves: a critical-state finite element model matched peak capacity but gave drained displacements to peak of only 45 to 55 % of the measured values (Liu 2026, PDF pp. 138 and 143), and the thesis calls for calibrating "the parameters governing the stiffness response and its degradation with strain" (PDF p. 304).

9.6 Repeated loading. A one-way (monotonic) equation describes first loading only, but that is exactly the input needed by the only accumulation law for plate anchors in sand found in the sources

read. For the team: present the equation as the first-loading (backbone) curve that feeds an accumulation law; do not treat the displacement to peak as a displacement limit under cycling; keep the drained scope. In Kurniadi's method the displacement at the first cycle "is taken as the monotonic normalised anchor displacement at load Q_{max} " (Kurniadi 2024, pp. 129 to 130), and later displacement is scaled from it: $(\delta_N - \delta_1)/(\delta_1 f_{ampl} f_p) = 0.0755[\ln(1 + 0.31N) + 4.93 \times 10^{-5} N]$, with f_{ampl} a function of the cyclic amplitude and f_p of the mean pressure (pp. 119 to 125). It predicted two multi-amplitude tests not used in its calibration within 10 % (p. 131). In those centrifuge tests (saturated dense sand, 0.9 m prototype plate, H/D 3), 5,000 drained cycles with a maximum load of $0.9 Q_u$ carried the plate to δ/D 0.152, twice the monotonic displacement to peak of 0.07, without failure; post-cyclic capacity was 0.94 to 1.14 Q_u , and amplitude mattered more than mean load (pp. 111 to 127; see also Kurniadi et al. 2026). Under partial drainage the same rig failed during cycling at maximum loads of $0.8 Q_u$ (after 382 cycles) and $0.9 Q_u$ (after 39 cycles) (pp. 191 and 193), so there the one-way curve is not a safe bound. After cycling, anchors reload stiffer: on a keyed plate in dry dense sand, post-cyclic capacity rose by about 13 % (Chow et al. 2015, accepted manuscript, pp. 6 to 7), and the cyclic stiffness of helical anchors at 1 g settled at 3 to 9 times its first-cycle value, with post-cyclic capacity between 0.76 and 1.01 of the monotonic value (Hao et al. 2022, pp. 8 to 9 and 21).

9.7 Installation. No plate anchor curve read here compares an installed plate with a pre-placed one.

For the team: state that the equation describes pre-placed plates, and that installed plates may be softer at the start of loading. Apart from the keyed plates of Chow et al. (2020), which were pushed into the sand at 1 g before the centrifuge was spun up, every plate curve read used a plate placed while the sand bed was prepared; Chow et al. note that "the installation at 1g may affect the initial stiffness of the anchor load-displacement curve" (accepted manuscript, p. 13). For helical anchors the effect is large: an anchor installed in flight in the centrifuge carried 93 kN against 191 kN for one placed before spinning, and the curves separated after about 2.5 mm (Pérez et al. 2018, Fig. 8), and discrete element models put the change in tension stiffness from installation method at 43 %, up to 120 % in loose sand (Sharif et al. 2021, submitted version).

9.8 Load direction and level on a floating wind anchor. Mean loads are slow and drained; storm loads arrive at wave periods; shared anchors are loaded mostly along the weather direction.

For the team: report the curve at two points, near the mean load (drained) and near the design load, and state that loads out of the plate's plane or changing direction are outside the scope. Design tension combines a mean part and a dynamic part at wave periods of about 7 to 15 s (Fontana et al. 2019, Table 1; TAILWIND project report D3.1, 2025, pp. 22 to 23). For anchors shared between turbines, the resultant stayed within 10° of the wind, wave and current direction in 91 % of cases and its maximum fell by 0 to 25 % (Fontana et al., p. 5, Table 4), although one design study found sharing raised the design tension and flattened a taut line from 23.7° to 13.6° (TAILWIND D3.1, pp. 22 to 24). No mean-to-maximum ratio for catenary or semi-taut moorings was found in the sources read; the only pair is a tension-leg example with 8 MN pretension and a 16 MN maximum (Cerfontaine et al. 2023, p. 4).

What to record for every curve:

- the pull angle at the plate, and the plate's orientation;
- where displacement was measured (plate, or line or actuator);
- drainage: dry, saturated and drained (V below 1), or partially drained (exclude);
- stress level ($\gamma'H/p_a$, plate size and g level) and the sand's relative density or state parameter;
- H/B , and how the peak was defined (true peak, plateau, tangent-stiffness rule or a fixed δ/B);
- installation (pre-placed or installed) and loading (first, one-way loading).

10. Gaps the capstone can fill

1. **No fitted pre-peak equation for horizontal plates pulled upward in sand.** None of the twelve plate anchor papers read in full fits one; the closest equations are for spread foundations, piles and pipes, and the only fitted plate anchor equation read is Ghaly's (1997) power law for vertical plates pulled sideways, from 12 small laboratory tests (section 5), which Das and Seeley (1975) had fitted with a hyperbola (reported by Ghaly; not opened). Two 1981 papers that may fit hyperbolas under repeated loading could not be opened. This is a statement about the papers read, not a systematic search.
2. **The onshore uplift curve stops at an embedment ratio of three.** Trautmann and Kulhawy (1988, p. 180) exclude "deep plates, anchors"; Phoon et al.'s spread foundations run to D/B 3.7 (p. 3). Offshore plates and most anchor tests beyond H/B 3 (Dickin, Ilamparuthi, Kurniadi) are outside both.
3. **No construction from elastic start, capacity and fitted shape.** Rowe and Booker (1979) give K_0 , existing methods give Q_u , and section 3.3 shows what data must add; no opened paper cites Rowe and Booker for this. Mobilisable strength design has not been applied to anchors in sand in anything screened here.
4. **Pooling without a stress-level variable.** Dickin's factor of three between 1 g and centrifuge failure displacements means a pooled 1 g dataset will understate field displacement unless test type is a variable. Matching the state parameter is one tested way to bring 1 g tests closer to centrifuge results, particularly at greater depth (Liu 2026, PDF p. 300).
5. **No anchor serviceability check in the guidance opened,** which leaves the team to set its own working-load criterion (for example the displacement at $Q/Q_u = 0.5$ or at the design load). Read here up to DNVGL-ST-0119 (2018) and API RP 2SK (2005); the 2025 DNV edition and the 2024 API edition were not available.
6. **Line against plate displacement.** Keyed-anchor tests report mooring-line displacement (Chow et al. 2015); a plate-displacement equation needs those curves converted or excluded.
7. **Installed against pre-placed plates.** No plate anchor test read measures what installation does to initial stiffness (section 9.7).
8. **The finite element models read are too stiff before peak in drained loading.** A NorSand model that matched capacity gave 45 to 55 % of the measured drained displacement to peak, though undrained displacements agreed (Liu 2026, PDF pp. 138 and 143), and in Roy et al.'s (2022) consolidation study measured displacements were about 10 to 12 times the simulated ones; so measured curves remain the basis for the shape, which supports the team's choice of published experimental data.
9. **Measured curves at angle and depth.** Among the papers read, no centrifuge test reports a horizontal plate under an inclined pull against plate (not line) displacement, and measured deep curves (H/B above 6) come from two centrifuge series, Dickin (1988, to H/B 8) and the Hao et al. (2019) tests tabulated by Kurniadi (2024, to H/D 12) (sections 4, 9.1, 9.3 and 9.5).

Reading basis

Work	Used for	Read
Ilamparuthi et al. 2002; Dickin 1988; Murray and Geddes 1987; Liu et al. 2012; Al Hakeem and Aubeny 2019; Kurniadi et al. 2026 (JGGE)	Data, density, scale, cyclic	Full text, published
Sakai and Tanaka 1998; Tagaya et al. 1988	Data, scale (Tagaya: no curves found)	Full text, scans read from page images (Tagaya surveyed at reduced size)
Giampa et al. 2019; Bradshaw et al. 2016	Data	Full text, typeset copies, version unconfirmed
Zhuang et al. 2022; Roy et al. 2021 (CGJ)	Data; capacity database	Full text, submitted manuscripts

Work	Used for	Read
Trautmann and Kulhawy 1988; Phoon et al. 2007; Trautmann et al. 1985; Cheuk et al. 2008; Rowe and Booker 1979	Equations, mobilisation, elastic stiffness	Full text, published (Rowe and Booker chart values read by eye)
Osman and Bolton 2005; Vardanega and Bolton 2011	Mobilisable strength design, power law	Full text, published
Rahimi et al. 2024; Reddy and Stuedlein 2017; Cerfontaine et al. 2020	Pile and screw anchor curves	Full text, accepted manuscripts
Cerfontaine et al. 2023	Mooring context	Full text
Chow et al. 2015	Cyclic, pooling trap	Full text, accepted manuscript
Chow et al. 2020	Drainage	Abstract
Udrescu and Tegmark 2020; Brunton et al. 2016; Cranmer 2023; Wang et al. 2022	Methods	Full text (Brunton and Cranmer as arXiv versions)
DNV-RP-E302 (2002), DNV-OS-E301 (2013), DNV-OS-J103 (2013), DNV-RP-F110 (2007)	Guidance	Full text, clauses cited
DNVGL-ST-0119 (July 2018)	Guidance	Full text (163 pages); keyword search plus Sec. 1, 5.1.3, 9.1, 9.7 read
API RP 2SK, 3rd edition (October 2005)	Guidance	Scanned copy; OCR of section 7 and Appendix E (PDF pages 38 to 39, 101 to 126)
Kepmen ESDM 1846 K/18/MEM/2018; RSNI 3 draft of SNI 8460 (April 2017)	Indonesian guidance	Full text (decree by OCR, pp. 1 to 70; SNI as draft)
API RP 2SK, 4th edition (2024); ISO 19901-7 (2026)	Guidance	Scope only
Ghaly 1997, with its discussion, closure and erratum	Sideways power law (section 5)	Full text, published; equation read from page images
Hanna et al. 2014; Tilak and Samadhiya 2023; Dickin and Leung 1983; Cheng et al. 2025; Lee et al. 2014; Neubecker and Randolph 1995; Pérez et al. 2018; Hao et al. 2022	Angle, line, scale, installation, cycling (section 9)	Full text, published
Roy et al. 2022 (inclined loading); Zhuang et al. 2022 (Table 2, Fig. 3)	Angle, line displacement	Full text, submitted manuscripts
Roy et al. 2022 (consolidation); Chow et al. 2020	Drainage, keyed plates	Full text, accepted manuscripts; Chow et al. corrigendum published
Kwa et al. 2025; Sharif et al. 2021	Line rotation; installation	Full text, accepted or submitted manuscripts, passages read
Kurniadi 2024, PhD thesis (University of Melbourne)	Deep plates, cyclic accumulation	Chapters 3, 4 and 7, passages read
Liu 2026, PhD thesis (University of Melbourne and University of Toronto)	Drainage, scaling, numerical stiffness	Abstract; PDF pp. 138, 143, 263 to 264, 297 to 305
Schiavon 2017, PhD thesis (University of São Paulo)	Andreadis 1981 description; helical cyclic	pp. 50 to 53, 124 to 125, 140, 190 to 191
Fontana et al. 2019; TAILWIND D3.1 (2025)	Load direction and level	Full text, published

Not opened, and not used for any claim: Rowe and Davis 1982, White et al. 2008, Roy et al. 2021 (Géotechnique), Oztoprak and Bolton 2013, Fleming 1992 and Bolton 1986 (Emerald gave the abstract page only); Kondner 1963 and Hardin and Drnevich 1972 (no ASCE access); Stuedlein and Uzielli 2014; Dickin and Leung 1983; Hsu and Liao 1998; Chow et al. (Ocean Engineering) and Kurniadi et al. 2025 (Ocean

Engineering) (Elsevier); Bradshaw et al. 2015; DNV-ST-0119 (2025) and the other current DNV editions (subscription). For section 9: Andreadis et al. 1981 and Andreadis and Harvey 1981; Chow et al. 2018 and Han et al. 2022 (dynamically installed plates); Al Hakeem and Aubeny 2021 (keying); Roy and Chow 2022 and Cheng et al. 2022 (inclined plates); Rui et al. 2022 (chain); Hong et al. 2025 (combined loading); Xiao et al. 2026 (inclined helical anchors); Lee et al. 2019; Kurniadi et al. 2023 and Rasulo et al. 2017 (OSIG); Bemben and Kupferman 1975 and Colp and Herbich 1975 (OTC); O'Loughlin and Barron 2012; Schiavon et al. 2017 and 2018; Murray and Geddes 1989; Choudhary and Dash 2018; the theses of Barron 2014, Stewart 1988 and Giampa 2014 and 2017; Thorel et al. 2020; and a 2023 OSIG paper on horizontal plate anchor capacity in dilating sand (10.3723/xvjq7767), which is directly on the team's topic.

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