Electronic supplementary materials
for https://doi.org/10.1631/jzus.A2200446

Monotonic uplift behavior of anchored pier foundations in soil overlying rock

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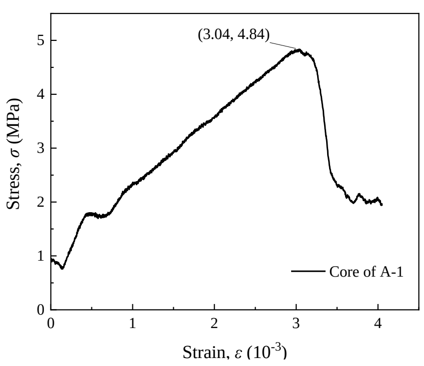
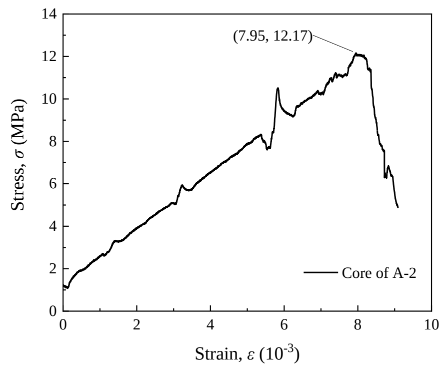
S1 Classification of core samples

Table S1 Classification of cores according to the Rock Mass Rating System (RMR) (ASTM D5878)

Parameters		Values		
		A-1	A-2	A-3
Strength	Point-load		<1.00 MPa	
	Uniaxial compressive strength	4.84 MPa	12.17 MPa	1.37 MPa
Drill core quality RQD			25% - 50%	
Spacing of discontinuities			60–200 mm	
Condition of discontinuities		Gouge < 5 mm thick	Gouge < 5 mm thick	Separation > 5mm
Ground water			None	
Rating adjustment for joint orientations (Foundations)			-7	
Ratings		35	36	25
Class number and description			IV, Poor rock	
Cohesion of the rock mass			100 - 200 kPa	
Friction angle of the rock mass			15 °- 25 °	



(a)



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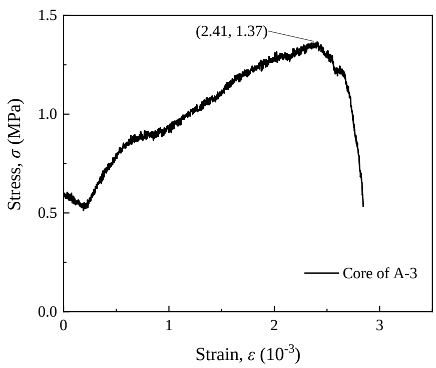


(c)

Fig. S1 Stress-strain curves of core unconfined uniaxial compression tests: (a) A-1; (b) A-2; and (c) A-3

S2 Estimation of deformation modulus of soil and rock

The dilatometer modulus E_D is proposed in the theory of elasticity by Gravesen (1960), and its relationship with deformation modulus is:

$$E_s = FE_D \tag{S1}$$

where E_s is the deformation modulus of the soil, F is an empirical coefficient related to the material index I_D (Marchetti, 1980), and F can be 2 for sand (Campanella and Robertson, 1991).

Marchetti and Crapps (1981) proposed an empirical chart that can be used to estimate the dilatometer modulus E_D of soil in the research of flat dilatometer, as shown in Fig. S2. The dilatometer modulus E_D is defined as a quantity calculated from the field readings of the flat dilatometer (Gravesen 1960), which cannot be regarded as the deformation modulus of the soil. The abscissa in the chart, the material index I_D, is also defined as a quantity calculated from field test data. According to Marchetti and Crapps (1981), the soil type can be identified as follows:

clay $0.1 < I_D < 0.6$

silt $0.6 < I_D < 1.8$

sand $1.8 < I_D < (10)$

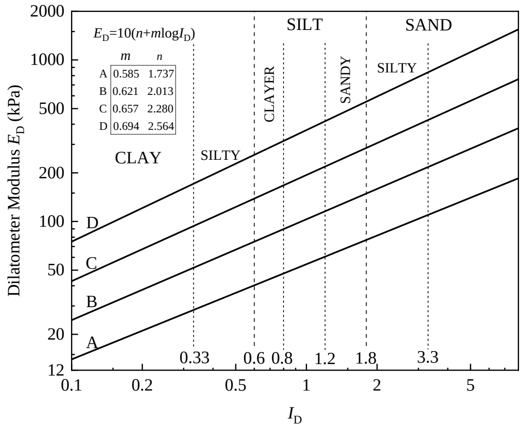


Fig. S2 Chart for estimating soil dilatometer modulus

The covering soil in this test is gravel sand, which belongs to sand with high I_D and modulus. The dilatometer modulus of this sand is determined to be in the range of 500 kPa to 2000 kPa. Due to the lack of other detailed survey or test data as a reference, the dilatometer modulus is estimated to be 1000kPa. The estimated deformation modulus of the soil is 2000 kPa.

The deformation modulus of the rock, E_r, can be estimated by RMR through the transformation model proposed by Gokceoglu et al. (2003), as shown in Eq. (S2).

$$E_{\mathrm{r}} = 0.0736e^{0.0755\mathrm{RMR}} \tag{S2}$$

The estimated deformation modulus of the rock mass at A-1, A-2 and A-3 are 1034 MPa, 1115 MPa and 486 MPa, respectively.

According to the observation of the core samples (Fig. 3b) and the construction, the piers were embedded in the rock about 0.5 m. Therefore, the deformation modulus of the mass around the pier needs to consider the influence of rock socketing. The estimated soil layer modulus cannot be directly used to calculate the uplift deformation of the pier. The calculation method of the layered soil modulus can be used here, as shown below:

$$E_{\mathrm{s,eq}} = \frac{E_{\mathrm{s1}}h_1 + E_{\mathrm{s2}}h_2 + \cdots + E_{\mathrm{sn}}h_{\mathrm{n}}}{h_1 + h_2 + \cdots + h_{\mathrm{n}}} \tag{S3}$$

where $E_{s,eq}$ is the weighted average deformation modulus of the mass around the pier, E_{sn} and h_1 are the deformation modulus and thickness of each layer, respectively. The deformation modulus of the mass around the piers at A-1, A-2 and A-3 is estimated to be 174 MPa, 188 MPa and 83 MPa.

S3 Interpretation of ultimate uplift capacity

Some of the recognized criteria for defining the ultimate uplift capacity (UUC) of the foundation are the mathematical modelling (Chin, 1970), limit displacement method (De Beer, 1970), and graphical construction (Tomlinson, 1977; Kulhawy and Hirany, 1989).

As a graphical construction method, the "tangent intersection method" is intuitive and practical to determine the UUC, suitable for analysing the test results. An example of the application of this method is shown in Fig. S3. Firstly, the uplift *P-s* curve can be simplified into three distinct regions: the initial linear region (OL_1), the transition region (L_1L_2), and the final linear region (L_2L_3) (Hirany and Kulhawy, 2002). Secondly, the ULS is determined as the intersection of two lines drawn as tangents to the initial linear and final linear portions of the *P-s* curve and projected to the *P-s* curve. The load at the ULS is the UUC of the foundation. In addition, the inflection point of the *P-s* curve can be directly determined as the ULS (ASTM D3689-90).

Point L_1 (elastic limit) corresponds to the load P_{L1} (elastic limit load) and displacement s_{L1} (elastic limit displacement) at the end of the initial linear region (OL_1), while point L_2 (failure threshold) corresponds to the load P_{L2} (interpreted failure load) and displacement s_{L2} (interpreted failure displacement) at the initiation of the final linear region (L_2L_3) (Chen, et al. 2008). L_3 indicates the final loading state. Hirany and Kulhawy (1988) suggested that, for uplift loading of drilled shaft foundations, L_2 occurred at a mean displacement of about 13 mm (s_{L2}) and that L_1 occurred at about 0.4% of the shaft diameter (s_{L1}). For foundations with experimentally measured *P-s* curves, s1 and s2 can be determined according to the tangent method introduced above, i.e., s_{L1} corresponds to the end of the initial linear region, and s_{L2} corresponds to the initial place of the final linear region.

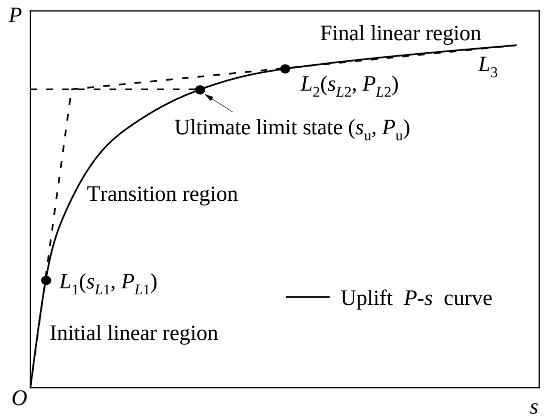


Fig. S3 The characteristic region of the uplift P-s curve and the determination method of the ULS

S4 Calculation procedure for UMC of anchored pier foundation

According to the calculation method of uplift mobilization coefficient (UMC) proposed in this study, an example of the calculation procedure of the UMC of foundation A-1 is given. First, the preliminary design of foundation A-1 was carried out according to the specifications (BSI 2000; IEEE 2001) and related studies (Serrano and Olala, 1999; Das, 2017), within the constraints of geology and project requirements. Then, the uplift capacity of piers and anchor groups of foundation A-1 was calculated based on geological parameters and specifications. The UUC of the pier and anchor group of A-1 measured by strain gauges (P_{pu} = 614 kN, P_{au} = 524 kN) were directly used in the computational analysis process in this study. By substituting the geometry of A-1 (Table 1) and the geological parameters at A-1 (Table 2) into Equations (5) and (7), K_p = 1482 kN/mm and K_a = 1006 kN/mm can be calculated. Then the UMC calculation was carried out, assuming that k_1 = 1 and calculating k_2 for A-1 yielded 0.83. The results of k_1 and k_2 were satisfying the qualification ($k_1 \leq 1$ and $k_2 \leq 1$). The above results were substituted into Equation (2) to calculate the DUC of A-1, which was 1059.4 kN. The calculation procedure of UMC is shown in Fig. S4.

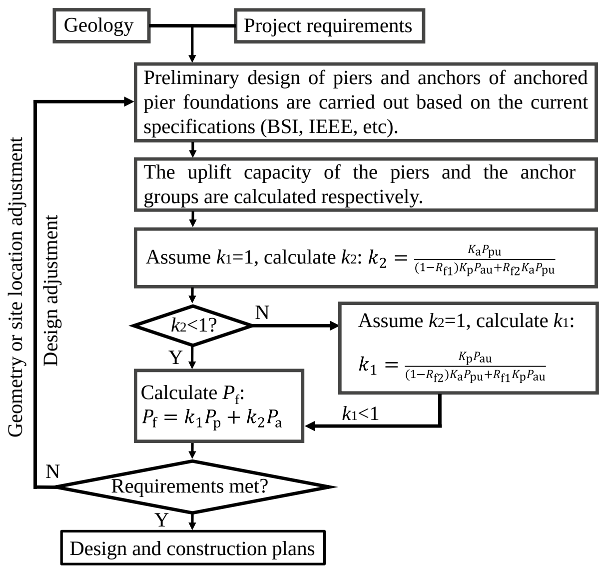


Fig. S4 Calculation procedure for UMC of anchored pier foundation

S5 Test information of the calculation cases

The calculation cases include foundation A-1, A-2 and A-3 of this study and other test cases (Cheng et al. 2012; Qian et al. 2015; Ismael et al. 1979). To further improve the reliability of verification, the pier GTC2 and anchor group AG8 reported by Qian et al. (2015) and Ismael et al. (1979) respectively were combined into an anchored pier as a calculation case. Fig. S5 demonstrates the *P*-*s* curves of CF-A, CF-B and GTC2+AG8. The dimensions and geological parameters of all cases are shown in Tables S2 and S3, where c is the cohesion, φ is the friction angle, and γ is the unit weight.

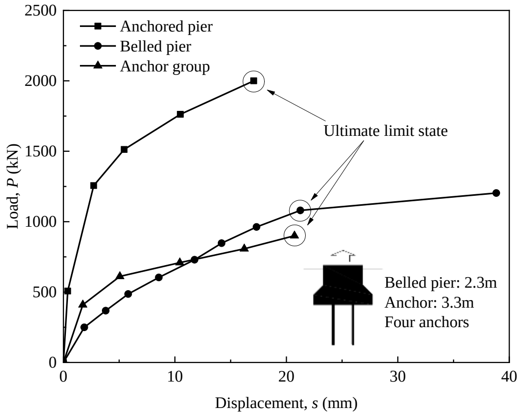
Table S2 Dimensions of test foundations in this study and other cases

Foundation	Type	L_p(m)	L_a(m)	d_1(m)	d_2(m)	d_a(mm)	d_b(mm)	Number of anchors
A-1	Anchored pier	3	1	1	1.5	120	40	4
A-2		3	2	1	1.5	120	40	4
A-3		3	3	1	1.5	120	40	4
CF-A		2.3	3.3	1	2.1	110	48	4
CF-B		2.8	3	1	2.3	110	48	4
GTC2+AG8		2.65	0.9	1.2	1.64	76	51	8

Table S3 Geology of this study and other cases

Foundation	Geology	
	Soil layer	Rock layer
A-1 A-2 A-3	Gravel sand with a thickness of 2.5 m, φ=20-25 °, γ=22 kN/m^3; the deformation modulus of the soil layer at A-1, A-2 and A-3 were about 174 MPa, 188MPa and 83MPa respectively.	Strongly weathered tuff with compressive strength 1.37-12.17 MPa, Class IV poor rock, φ=15-25 °, γ=23 kN/m^3; the deformation modulus of the rock at A-1, A-2 and A-3 were about 1034 MPa, 1115 MPa and 486 MPa respectively.
CF-A	Silt with a thickness of 2.3 m, c=26.9 kPa, φ=37 °, γ=18.8 kN/m^3; the deformation modulus was about 1MPa.	Strongly weathered sandstone, Class V very poor rock, RMR≈20, the deformation modulus was about 333 MPa.
CF-B	Gravel clay with a thickness of 1.6m, c=18.6 kPa, φ=28 °, γ=19 kN/m^3; the deformation modulus was about 209 MPa.	Strongly and Moderately weathered slate, Class IV poor rock, RMR≈25, the deformation modulus was about 486 MPa.
GTC2+AG	Gravel with cobbles (roughly classified	Slightly weathered shaly limestone with

8	as strongly weathered rock, Class V very poor rock), c<100 kPa, φ<15 °, γ=14.2-22.4 kN/m^3; the deformation modulus was about 107 MPa.	average shear value 172 kPa and compressive strength 101.1 MPa, Class III fair rock, RMR≈45, the deformation modulus was about 2200 MPa.



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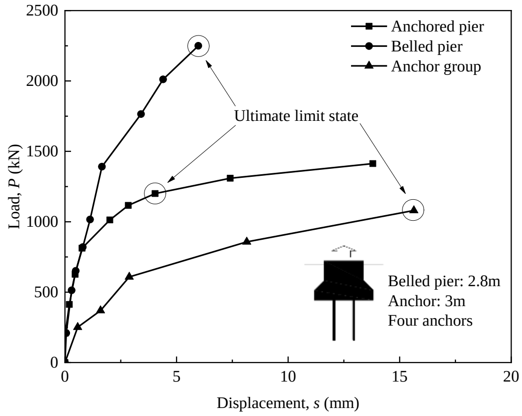

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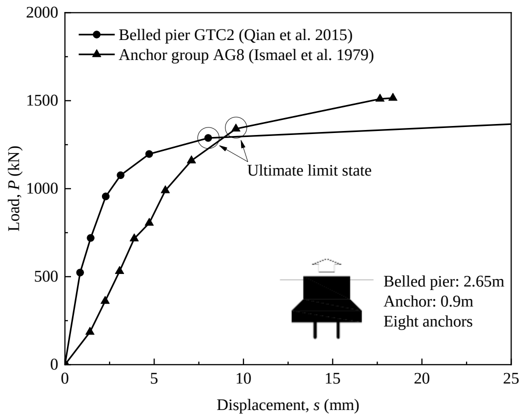

(c)

Fig. S5 *P-s* curves of the other cases (Cheng et al. 2012; Qian et al. 2015; Ismael et al. 1979): (a) CF-A; (b) CF-B; and (c) GTC2+AG8