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Estimation of strut forces for braced excavation in granular soils from numerical analysis and case histories

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Abstract: Construction work of many projects in urban areas requires deep braced excavations, under which circumstances, appropriate design method to assure both the safety and economy of the retaining walls and supporting systems is therefore of paramount importance. For multi-propped retaining walls, Apparent Pressure Diagram (APD) method developed by Peck (1969) is commonly adopted to determine the forces acting on the struts. Unfortunately, most previous studies focused on the performance of braced excavations in soft to stiff clays, where limited publications involve braced excavations in dense sand and gravel with medium rigid walls. In this study, both 2D and 3D finite element analyses were carried out to examine the strut forces for braced excavations in granular soils. Subsequently, based on the numerical results as well as the field measurements from a number of reported case histories, empirical charts have been proposed to determine strut loads for deep excavations in granular soils with medium rigid walls.

Keywords: Apparent Pressure Diagram, braced excavation, granular soils, strut forces, finite element analysis, medium rigid walls

1. Introduction

Deep excavations are increasingly carried out in urban areas with development of underground space. However, the excavation process inevitably alters the stress states underground and may introduce significant wall deformations and ground movements, which would cause potential damage to the adjacent properties. To reduce the excavation-induced deformations, appropriate retaining walls and supporting systems should be designed through adequate construction methods. Based on the Nicoll highway collapse (John 2013) in Singapore and the Xianghu foundation pit accident (Gong and Zhang 2012) in Hangzhou, China, an important design issue is to ensure the reliability of the structural systems, which is composed of retaining walls and struts. However, most previous studies (Chang and Wong 1996, Ou 2006, Sze 2010, Zhang and Goh 2013, Zhang et al. 2015, Do et al. 2016, Goh et al. 2017a, 2017b, Zhang et al. 2018a, 2018b, 2018c, Zheng et al. 2018) focused on performances of braced excavations in soft to stiff clays. There are only limited studies involving braced excavation in granular soils.

For multi-propped retaining walls, the Apparent Pressure Diagram (APD) method developed by Terzaghi and Peck (1967) and Peck(1969) is generally adopted to determine the forces acting on the struts. Different APDs for braced excavations in sands, soft to medium clays and stiff fissured clays were proposed respectively, based on the filed measured data.

Juran and Elias (1987) proposed a modified apparent pressure diagram based on in-situ construction data in sand. Their study indicated that the shape of the APD is almost the same as the stiff clay proposed by Terzaghi and Peck (1967), as shown in Figure 1a. Other APD for dense sand and loose sand provided by Terzaghi and Peck (1967) are shown in Figure 1b and 1c, respectively.

(a) stiff clay

(b) dense sand

(c) loose sand

Fig.1 APD for design of struts in sand and stiff clay (Replot based on Terzaghi and Peck, 1967)

Based on a database of the strut loads of eleven cases of excavations in granular soils from the USA, the UK, Brazil, Germany, and Japan, Twine and Roscoe (1999) proposed that the APD in sand can be calculated as:

$$p = K_a \gamma H \quad (1)$$

where p is the distributed prop load (in kN/m), K_a is the coefficient of lateral earth pressure and equals to 0.2 in dense sand, γ is the soil unit weight (in kN/m³), and H is the depth of the excavation (in meter).

Hsu (2014) summarized a number of twenty-five deep excavation cases in gravel formation in Taichung and obtained the distributions of APD at excavation depths of 15 m, 20 m, and 27 m. His research demonstrated that the apparent earth pressure is much less than that of Peck's APD. The coefficient of lateral earth pressure K_a , for H less than 15m, between 15m and 20m, and greater than 20m, are 0.07, 0.09, and 0.11, respectively.

With the rapid development of the computational capacity, different numerical methods such as finite element method (FEM) has been adopted to investigate the behavior of braced excavations. Hsiung et al. (2016) carried out FE analysis for the performance of braced excavations in loose to medium dense sand. The consistency of the results between measured data and the predicted values validates the reliability of FE analysis. It has been pointed out that the findings in their study generally agree well with the distributed prop load (DPL) suggested by Twine and Roscoe (1999) as well as the APD for sand suggested by Peck (1969) at depths from 3 m to 9 m.

Chee (2014) firstly compared the strut forces obtained from numerical analysis with the semi-empirically derived APD for braced excavations in sand and investigated the influences of sand friction angle and stiffness on strut forces. The results indicate that the APD for sand generally overestimates strut forces for the top two levels while underestimates the loads of the lower levels. Increasing the sand stiffness alone did not lead to significantly lower strut forces. Nevertheless, increasing the sand friction angle, which represents a higher stiffness, can result in significantly lower strut forces.

Apart from the numerical analysis, model test has been conducted by Nakai et al. (1999) and the results were compared with FE analysis. It was found that the computed results agreed well with the model test results.

As discussed previously, the determination of strut forces for braced excavation in granular soils is still hot issue due to limited research and the scarceness of reliable in situ monitoring data. The main purpose of this paper is to propose the updated APD for medium rigid wall systems based on extensive plane strain and three-dimensional numerical analyses supplemented by additional measured data from case histories. The influential factors on APD include the granular thickness,

soil strength and stiffness, wall embedded depth and wall rigidity. Consequently, three types of granular soil types, i.e. medium dense sand, dense sand and gravel with different geometrical properties are considered in this study. Some differences were observed between the numerical results and estimations from Peck's APD.

2. Numerical modelling

The FE software PLAXIS 2D (2017) and PLAXIS 3D (2017) were used to perform the excavation simulations. Fig.2 shows a typical cross-section and plan view for the cases considered. The symbol x represents the horizontal distance from the center of the excavation. For each level of struts, the struts at $x=-2$ m and $x=2$ m sections (when horizontal strut spacing was 4 m) are identical due to symmetry of the struts and walers. Fig.2 also plots the embedded retaining wall together with a five-level strut system for $H_e = 17$ m. H_p refers to the wall's penetration depth, other relevant design parameters shown in Fig.2 are summarized in Table 1.

Fig.2 Cross-section and plan view of the numerical model for braced excavation

97

Table 1 Parameters of geometrical properties

Parameter	Name	Geometrical properties
excavation length (m)	L	36,60,84 *
excavation width (m)	B	30
thickness of granular layer (m)	T_1	25
thickness of stiff clay layer (m)	T_2	30
horizontal strut spacing (m)	S_H	4,5
vertical strut spacing (m)	S_V	3
depth of final excavation (m)	H_e	17

* L=36m, 60m, 84m refers to L/B=1.2, 2.0, 2.8, respectively. For each L/B, two types of wall with four different stiffness values are considered.

98 **2.1 Numerical models**

99 For 2D analysis, only half of the excavation model was developed due to symmetrical conditions
100 for both the excavation sequence and geometry. A fine mesh size was adopted for 2D analysis to
101 improve the accuracy of calculations. For 3D analysis, only a quarter mesh and a medium mesh
102 size were used to reach a balance between accuracy and the processing time. Fig. 3 shows a
103 typical 3D mesh plot, comprising of 93713 nodes and 63343 15-noded wedge elements.

104

105 **Fig.3 Typical quarter 3D mesh of excavation**

In this study, the embedded retaining wall is simulated using 5-noded elastic plate elements for 2D analysis, while for 3D analysis, the wall is simulated using 8-noded quadrilateral plate elements with six degrees of freedom per node. Considering that the relatively flexible wall was generally designed and constructed for braced excavations in the competent granular soil in engineering practice, two types of wall with four different stiffness values were considered for each soil type, as listed in Table 2. A stiffness coefficient α was utilized to represent walls with varying rigidities (Bryson and Zapata-Medina 2012). The baseline bending stiffness EI used to develop the different models was $540,675 \text{ kNm}^2/\text{m}$, which refers to a wall of medium rigidity based on databases of Long (2001) and Moormann (2004). Therefore, $\alpha=1.0$ represents the cases with medium rigid wall in this study. For flexible walls in granular soils, the baseline bending stiffness EI of $540675 \text{ kNm}^2/\text{m}$ was multiplied by smaller α values of 0.06, 0.1, and 0.2. Meanwhile, based on method adopted by Finno et al. (2007), the wall thickness of 0.42 m was set to be constant so that the moment of inertia I and area A of the plate were kept constant, and only elastic modulus E was varied. Besides, the soil-structure interactions are simulated through interfaces on both sides of the wall, which allow for the specification of a reduced friction between wall and the soil.

Table 2 Wall properties for 2D and 3D analyses

Parameters	Wall types			
	flexible			medium
	Plane strain (2D) FE parameters			
α	0.06	0.1	0.2	1.0
Wall stiffness EI (kNm^2/m)	3.224×10^4	5.407×10^4	1.081×10^5	5.407×10^5
Compressive stiffness EA (kN/m)	2.206×10^6	3.678×10^6	7.356×10^6	3.678×10^7
Poisson's ratio, ν	0.15	0.15	0.15	0.15

Three-dimensional (3D) FE parameters					
Young's Modulus (kPa)	E_1	5.252×10^6	8.754×10^6	1.751×10^7	8.754×10^7
	E_2	2.626×10^5	4.377×10^5	8.754×10^5	4.377×10^6
Shear Modulus (kPa)	G_{12}	2.627×10^5	4.378×10^5	8.757×10^5	4.379×10^6
	G_{13}	8.754×10^5	1.459×10^6	2.918×10^6	1.459×10^7
	G_{23}	2.626×10^5	4.377×10^5	8.754×10^5	4.377×10^6
Poisson's ratio	ν	0	0	0	0

The shoring system of the braced excavation comprised of struts and walers. The struts were simulated via fixed end anchors in 2D analysis. For 3D analysis, beam elements were used to model the struts and walers, which have six degree of freedom per node. For the braced excavations considered in this study, the struts were designed horizontally at a regular spacing of 4 or 5 meters in two directions to form a strong frame. The walers transfer the forces from the retaining wall to the struts and also stiffen and align the wall. The properties of shoring system are tabulated in Table 3. □

Table 3 Properties of shoring system

Parameters		Struts	Walers
Material type		Elastic	Elastic
Young's Modulus E (kN/m ²)		2.1×10^8	2.1×10^8
Unit weight γ (kN/m ³)		78.5	78.5
Cross section area A (m ²)		0.007367	0.008682
Moment of inertia (m ⁴)	I_2 *	5.073×10^{-5}	3.668×10^{-4}
	I_3 *	5.073×10^{-5}	1.045×10^{-4}

* I2: Moment of inertia against bending around the second axis

I3: Moment of inertia against bending around the third axis.

The boundary conditions for the cases considered were: roller fixities at side boundaries to allow the vertical displacements; pinned at the bottom boundary to restrain any movements; the top boundary was free to move in all directions. For both 2D and 3D models, the lateral boundaries

in the side directions were defined as 90 m away from the center of the excavation to minimize the boundary effect of the mesh. The original ground water level inside the excavation was assumed at a depth of 5 m below the ground surface, which was progressively lowered with the excavation of the soil during each phase. Identical construction procedures of the simulation were applied as described in Table 4 for $H_e=17\text{m}$.

Table 4 Typical construction sequence for 2D analysis ($H_e=17\text{m}$)

Phases	Construction Details
Phase 1	Install the excavation wall
Phase 2	Excavate to 2 m below ground surface
Phase 3	Install strut system at 1 m below ground surface
Phase 4	Excavation to 5m below ground surface
Phase 5	Install strut system at 4 m below ground surface
Phase 6	Dewatering and excavation to 8m below ground surface
Phase 7	Install strut system at 7m below ground surface
Phase 8	Dewatering and excavation to 11m below ground surface
Phase 9	Install strut system at 10m below ground surface
Phase 10	Dewatering and excavation to 14m below ground surface
Phase 11	Install strut system at 13m below ground surface
Phase 12	Dewatering and excavation to 17m below ground surface

2.2 Constitutive model and model parameters

The hardening-soil (HS) model was used to simulate the constitutive behavior of the granular soils. This model involves frictional hardening characteristics to model plastic shear strain when subjected to primary deviatoric loading, and cap hardening to model plastic volumetric strain in triaxial compression loading conditions. Failure is still defined by the MC failure criteria. It should be noted that the hardening soil parameters for the sand and gravel considered in this parametric study were based on data extracted from Brinkgreve et al. (2010) and Halim (2008), respectively. The stiff clay with average undrained shear strength $c_u = 125 \text{ kPa}$ was based on the Gault clay found in Cambridge (Ng 1992). The HS parameters for different soils are summarized in Table 5.

Table 5 Hardening Soil Parameters Used for Finite-Element Modeling

Hardening soil parameter		Medium dense sand (drained)	Dense sand (drained)	Gravel (drained)	Stiff clay (undrained)
Parameter	Unit				
γ_{unsat}	kN/m ³	17	18.2	16	20
γ_{sat}	kN/m ³	19.8	20.3	20	20
$k_x = k_y = k_z$	m/day	1×10^{-8}	1×10^{-8}	1×10^{-4}	1×10^{-8}
E_{50}^{ref}	kN/m ²	30000	48000	60000	14847
E_{oed}^{ref}	kN/m ²	30000	48000	60000	14847
E_{ur}^{ref}	kN/m ²	90000	144000	250000	44540
C	kN/m ²	0	0	0	0.05
φ	°	34.3	38	44	33
ψ	°	4.3	8	14	0
v_{ur}	[-]	0.3	0.35	0.25	0.2
p^{ref}	kN/m ²	100	100	100	100
m	[-]	0.544	0.45	0.5	1
K_0^{nc}	[-]	0.436	0.384	0.305	0.455
R_f	[-]	0.938	0.9	0.9	0.96
R_{inter}	[-]	0.8	0.8	0.8	1

A total of 113 FE simulations using Hardening Soil (HS) model were carried out to investigate the influences of soil types, excavation geometries, wall stiffness α and strut level on the strut forces induced by excavation. Influence of wall's embedded depth on strut forces was investigated with typical FE models beforehand to choose the proper embedded-ratio H_p/H_e . For brevity, the numerical results of the study are presented with the main findings, as described in the next sections.

3. Analysis results

3.1 Effects of wall embedded depth on strut forces

To illustrate the influence of the wall's embedded depth on values of strut forces, sensitive analysis was performed with different ratio between wall embedded depth to excavation depth (H_p/H_e). Fig. 4 shows the typical cases in medium dense sand with $\alpha=1$, $L=36\text{m}$, $B=30\text{m}$, $T_1=25\text{m}$, $T_2=30\text{m}$, $S_H=4\text{m}$ and $H_e=17\text{m}$, H_p was assumed to be 17m, 15m, 13m, 11m, 9m, 7m and 5m, where corresponding embedded-ratio H_p/H_e equals to 1.0, 0.88, 0.76, 0.65, 0.53, 0.41 and 0.29, respectively. In Fig. 4, S1 denotes the first strut level and S5 represents the fifth level. As can be inferred from the diagrams illustrated in the figure, the general trend is that strut forces in strut S2~S4 decrease markedly when H_p varies from 29% H_e to 53% H_e , strut forces in strut S5 are noticeably varied when H_p/H_e rises from 0.29 to 0.76 and slightly varies when H_p/H_e greater than 0.76. Strut forces in strut S1 changes mildly when wall embedded depth varies from 29% H_e to 100% H_e , which is similar to the trend observed by Bahrami et al (2018). In this paper, H_p was fixed at the length of 13m (where H_p/H_e equal to 0.76) to study the effect of various wall rigidity and L/B ratios of strut forces in three types of soils.

Fig. 4. Variations of strut forces vs. increasing the wall embedded depth in medium dense sand

3.2 Numerical modeling results

In this section, the general trends of strut forces with varying wall rigidity and L/B ratios for the three granular soils are presented. As Fig. 2 depicted, the struts that are the closest to the center of the excavation among all the struts at the same level are subjected to the greatest force. Therefore, only the strut forces at location $x=2$ m (depicted schematically in red in Fig.2) are presented and analyzed.

The local and global instabilities of the models have already been checked and there is no ultimate limit state failure. In addition, the instabilities of the wall (such as sliding, overturning, rotating, etc.) and those relating to the struts, whether local or global, are supposed to be verified in all cases. For brevity, only some of the main results of the strut forces from the 2D and 3D analyses are presented in Tables 6 and 7. Generally, the struts S3, S4 and S5 are subjected to larger compressive strut forces. The general trend was that the axial forces in struts S2 to S5 decrease with the increase of soil strength while increase with decreasing wall stiffness. It can be also observed from Tables 6 and 7 that the differences of maximum strut forces depend on the granular soil types, L/B ratios, and the strut levels.

Table 6. Typical maximum strut force for $H_e = 17$ m and $\alpha = 1.0$ ($S_H=4$ m)

Soil type	Strut level	Maximum strut force (kN)			Max. difference between 2D and 3D (%)	
		2D	3D			
			$L/B = 1.2$	$L/B = 2.0$		$L/B = 2.8$
Medium dense sand	S1	186.22	203.24	191.22	203.15	9.14
	S2	740.53	799.82	673.24	769.94	8.01
	S3	1228.64	1363.31	1052.42	1321.16	10.96
	S4	1491.76	1706.41	1397.50	1678.91	14.39
	S5	1324.08	1589.24	1367.19	1537.54	20.03
Dense sand	S1	141.88	165.51	165.95	163.61	16.97
	S2	667.89	683.97	698.44	687.96	4.57
	S3	1046.53	1071.64	1144.05	1117.87	9.32
	S4	1247.63	1304.53	1410.36	1374.93	13.04

	S5	1129.53	1252.95	1338.17	1280.45	18.47
	S1	84.16	128.45	118.44	114.55	52.63
	S2	632.34	592.62	587.56	584.41	6.28
Gravel	S3	986.49	935.02	956.31	942.38	3.06
	S4	1188.79	1135.78	1174.26	1136.67	1.22
	S5	1051.24	917.33	1046.82	1003.39	0.42

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Table 7. Typical maximum strut force for $H_e = 17$ m and $\alpha = 0.06$ ($S_H=4$ m)

Soil type	Strut level	Maximum strut force (kN)				Max. difference between 2D and 3D (%)
		2D	3D			
			$L/B = 1.2$	$L/B = 2.0$	$L/B = 2.8$	
Medium dense sand	S1	148.89	176.42	181.17	182.97	22.89
	S2	928.36	1027.06	999.60	980.74	10.63
	S3	1372.98	1361.79	1404.45	1450.58	5.65
	S4	1611.11	1704.55	1703.64	1723.25	6.96
	S5	1698.13	1906.37	1918.09	1977.13	16.43
Dense sand	S1	174.69	220.79	219.29	224.22	28.35
	S2	877.85	971.27	939.49	898.32	10.64
	S3	1233.71	1263.22	1308.61	1248.18	6.07
	S4	1359.24	1402.10	1520.28	1436.74	11.85
	S5	1382.82	1850.74	1699.90	1567.95	33.83
Gravel	S1	125.72	174.92	176.16	177.04	40.82
	S2	751.95	693.12	679.06	691.98	7.82
	S3	1079.90	948.65	1023.13	1024.65	5.12
	S4	1230.48	1171.73	1211.93	1232.38	0.15
	S5	1605.14	1528.05	1424.34	1358.03	4.80

192

Fig. 5 presents the maximum strut forces of different strut levels for various L/B ratios in dense

193

sand ($\alpha = 0.06, 0.1, 0.2$ and 1). In general, the strut forces calculated by 3D simulations are

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greater than results from 2D simulations. It can be found from Fig. 5 that there are discrepancies

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from 2D simulations and 3D simulations at each strut level. However, the distribution patterns of

196

strut force obtained from different models are almost similar, i.e. the forces in struts S1 to S5

197

decrease with the increase of wall stiffness α . Meanwhile, as the wall stiffness increases from

198

$\alpha=0.1$ to $\alpha=0.2$ and then to 1.0 , the strut level with the greatest prop force changes from level S5

to S4. It is obvious from Fig. 6 that for a particular soil type, the maximum strut forces from S1 to S5 decrease with the increase of wall system stiffness. In addition, for the same wall rigidity, the maximum strut force decreases significantly from over 2000 kN for $\alpha=0.06$ of dense sand to less than 1400 kN for $\alpha=0.06$ of gravel. This again confirms that the strut forces depend on the soil type, the excavation geometries, the strut level, the wall rigidity and the resultant influences from these factors are complex.

Fig. 5 Maximum strut forces for various L/B ratios in dense sand ($\alpha = 0.06, 0.1, 0.2$ and 1)

Fig. 6 Maximum strut forces for various α in different soils ($L/B=2.8$)

4. Apparent Earth Pressure Diagrams

For design of braced deep excavation in soils, APD diagrams are generally employed to estimate the horizontal prop forces at different depths during excavation. It is widely accepted that the calculated earth pressure varies with the excavation depths. Herein the results of the 2D and 3D maximum strut forces for $H_e = 17$ m were normalized to derive the APD for excavations in medium rigid walls in this study. The tributary area load distribution procedure proposed by Peck (1969) was used. As an example to the case, Fig. 7 shows the assumed loading area of struts for $H_e = 17$ m, where h_{si} represents the loading area depth for the corresponding strut.

219

220 **Fig.7 Cross-section: assumed tributary loading areas of struts for $H_e = 17$ m**

221 For the strut force P , the apparent earth pressure σ_h can be obtained by:

222
$$\sigma_h = \frac{P}{S_H h_{si}} \quad (3)$$

223 The normalized depth (z/H_e) and the normalised apparent earth pressure ($\sigma_h/\gamma H_e$) are then plotted.

224 Only excavations in medium rigid walls ($\alpha=1$) for different granular soil types are considered in

225 view that the common use of these walls for deep excavations in such soil types in actual

226 projects. For comparison, Peck's APD, the CIRIA DPL recommendations (Twine & Roscoe

227 1999), as well as various case histories from the literatures as tabulated in Table 8, are plotted

228 together with the numerical results, for better comparison.

Table 8 Summary of case histories

Case histories	Soil type	Unit weight (kN/m ³)	H_e (m)	Wall type	No. of Strut levels	Horizontal strut spacing (m)	Vertical strut spacing (m)	References
New York (CIRIA report) C2	Gravel	17.7	8.6	KP (medium stiff wall)	3	3 6.1 6.1	3 2.2 2.2	Twine and Roscoe (1999)
Tokyo (CIRIA report) C9	Gravel	16.9	20.1	SP (medium stiff wall)	6	-	4.4 2.5 2.7 3 3 3.5	Twine and Roscoe (1999)
(CIRIA report) D13	Dense sand	19.6	21.3	KP (medium stiff wall)	4	3.6	5.3 4.6 4.6 4.5	Twine and Roscoe (1999)
Sao Paulo (CIRIA report) C8	Dense sand	20	18	KP (medium stiff wall)	5	-	3.5 3.2 3.1 3.4 3.3	Twine and Roscoe (1999)
Washington (CIRIA report) C7	Dense sand	18.9	18.3	KP (medium stiff wall)	5	4	2.9 3.4 3.3 3 3.6	Twine and Roscoe (1999)
Oakland (CIRIA report) C10	Dense sand	19.6	20.7	KP (medium stiff wall)	5	3.6	6.6 3.2 3 3.2 3.2	Twine and Roscoe (1999)

Munich (CIRIA report) C3	Gravel	19.6	11.9	KP (medium stiff wall)	4	2	2.4 3.1 2.7 2.5	Twine and Roscoe (1999)
(CIRIA report) D14a	Gravel	19.4	20	CC (medium stiff wall)	3	8.3	5.8 6.8 5.5	Twine and Roscoe (1999)
Toronto Transit Commission (TTC) subway structure	Dense sand	18~22	9~20	SP (medium stiff wall)	-	-	2.4~5.8	Boone and Westland(2004)
Taiwan, Taichung	Gravel	22.5	15-27		-	-	-	Hsu, et al. (2014)
(CIRIA report) C4	Dense sand	16.7-16.9	10.7-11.8	KP (medium stiff wall)	4	2	2.3 3.2 2.6 2.0	Twine and Roscoe (1999)
Singapore Serangoon Station	Dense sand	20	25	SP (medium stiff wall)	6	-	-	Couttsa et al.(2001)
Singapore Serangoon Station	Gravel	20	23.4	SP (medium stiff wall)	5	-	-	Couttsa et al.(2001)
Canada Water Station	Gravel	18.8	15.44	SP (medium stiff wall)	2	-	4 7	POWRIE and BATTEN(2000)
Notations: KP=King post wall; SP=Sheet pile wall; CC: Concrete contiguous pile wall								

230 The apparent earth pressure for medium rigid walls ($\alpha=1$) are shown in Fig. 8 for dense sand
231 and Fig. 9 for gravel, respectively. For cases with medium dense sand or flexible walls, due to
232 the limited case histories reported, the calculation results are not included in the plot. For dense
233 sand (Fig. 8), the dash line defines the upper bound of Peck's APD assuming the coefficient K_a
234 is equal to 0.2. Represented by the dark blue solid line is the characteristic distributed prop load
235 diagram of $DPL / \bar{\gamma} H_e = 0.2$ reported by CIRIA (Twine and Roscoe 1999). For gravel (Fig. 9),
236 due to the limited studies on strut forces in gravel formations, the line proposed by Hsu (2014)
237 was plotted, where the maximum prop load line was $0.11\gamma H_e$, as illustrated by the green dash
238 line.

239 For excavations in dense sand with medium stiff walls, as shown in Fig. 8, the numerical results
240 (represented by open symbols) indicate that the apparent earth pressure is generally less than
241 $0.1\gamma H_e$ for $z/H_e \leq 0.2$ while for $z/H_e \geq 0.2$, the apparent earth pressure is generally less than
242 $0.35\gamma H_e$. Based on the measured strut forces from the case histories (denoted as solid symbols)
243 listed in Table 8, for $z/H_e \leq 0.2$, the apparent earth pressure is generally less than $0.15\gamma H_e$, while
244 for $z/H_e \geq 0.2$, it is less than $0.3\gamma H_e$. Based on the numerical results and field measurements,
245 Peck's APD (which was developed based on medium rigid walls as well) underestimates the
246 strut forces, especially for the normalized depth from $z/H_e=0.2$ to $z/H_e=0.8$.

247 It can also be observed in Fig. 8 that the diagram proposed by Twine and Roscoe (1999)
248 overestimates the apparent earth pressure for $z/H_e \leq 0.2$ while underestimates it for $z/H_e \geq 0.2$.
249 Therefore, for braced excavations in dense sand with medium rigid walls, a threshold apparent
250 earth pressure of $0.15\gamma H_e$ for $z/H_e < 0.2$ and a threshold apparent earth pressure of $0.3\gamma H_e$ for $z/H_e \geq$
251 0.2 is proposed in this study, as indicated in Fig. 8, by the red solid lines.

Fig. 8 Normalized apparent earth pressure versus normalized depth for medium rigid walls in dense sand

For excavations in gravel with medium rigid walls, as shown in Fig. 9, the numerical results indicate that the apparent earth pressure is generally less than $0.1\gamma H_e$ for $z/H_e \leq 0.2$, while for $z/H_e \geq 0.2$, the apparent earth pressure is generally less than $0.35\gamma H_e$. Based on the measured strut forces from the case histories in Table 8, for $z/H_e \leq 0.2$, most of the data are less than $0.1\gamma H_e$, except for one point (which is $0.25\gamma H_e$). While for $z/H_e \geq 0.2$, the apparent earth pressure is also generally less than $0.35\gamma H_e$, while two points (with $0.39\gamma H_e$ and $0.40\gamma H_e$, respectively.) are slightly over $0.35\gamma H_e$.

Considering that the case histories summarized by Twine and Roscoe 1999 are not based on the pure gravel formations, the measured strut forces from these cases may sometimes deviate from

the mean values of the strut forces in the pure gravel formations. Thus, for multi-propped excavations in gravel formations with medium rigid walls, a conservatively threshold distributed prop load of $0.2\gamma H_e$ for $z/H_e < 0.2$ and a threshold distributed prop load of $0.35\gamma H_e$ for $z/H_e \geq 0.2$ is proposed, as shown in Fig. 9, by the red solid lines.

Fig. 9 Normalized apparent earth pressure versus normalized depth for medium rigid walls in gravel

Noted that APDs for braced excavations in medium dense sand are not discussed in present study for lack of relevant case histories. Table 9 summarizes the recommended APD in this study for braced excavations in dense sand and gravel using medium rigid wall systems, in the form of $K_a\gamma H_e$. The threshold distributed prop load of gravel is greater than that of dense sand for the scarceness of the pure gravel formation in case histories considered. However, it is found that the amount of calculated earth pressure varies with maximum excavation depth, and the

recommended K_a values for medium rigid walls in granular soils are mainly used for H_e less than 25m. For further study based on excessive case histories, different K_a values are suggested to propose for different wall rigidities and excavation depths in granular soils.

Table 9 Recommended K_a values for medium rigid walls in granular soils

soil type	K_a
Dense sand	0.15 ($z/H_e < 0.2$)
	0.30 ($0.2 \leq z/H_e \leq 1.0$)
Gravel	0.20 ($z/H_e < 0.2$)
	0.35 ($0.2 \leq z/H_e \leq 1.0$)

6. Summary and conclusions

In this study, a series of 2D and 3D FE simulations using the HS model for braced excavations in granular soils (sand and gravel) were carried out. It is concluded that for a particular soil type, the strut forces calculated by 3D simulations are generally greater than those from 2D. The general trend for strut forces is to decrease with increasing soil strength and increase with decreasing wall system rigidity. Combining the field measurements from a number of reported case histories and the numerical results, empirical charts for dense sand as well as gravel formations have been proposed for braced excavations with medium rigid wall systems as shown in Fig. 8 and Fig. 9, respectively, and the K_a values for medium rigid walls in granular soils are also summarized in Table 9.

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