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ORIGINAL PAPER

Geotechnical design parameter evaluation using the alluvial plain characteristics in southeastern Iraq

Jinung Do[1] · Ong Heo[2] · Yeo-Won Yoon[2] · Ilhan Chang[3]

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Abstract

Geotechnical construction is responsible for the overall stability of superstructures, and if there are design errors, the structure will be exposed to potential problems. Geotechnical design starts with the correct interpretation of the target ground. Southeastern Iraq is mainly comprised of an alluvial plain with diverse geological features, and, therefore, geotechnical design requires a detailed interpretation and understanding of the area. This paper reports on laboratory and field tests and in-depth analyses conducted on these alluvial plains. The results reveal that the upper layer of this area is highly over-consolidated. This may have been caused by the removal of overburden pressure as a result of glaciation and desiccation. The highly over-consolidated soils caused considerable sample disturbance by swelling the bored sample; this provided less reliable results. However, the cone penetration test was regarded as the most appropriate field assessment method for deriving sensible geotechnical design parameters. Despite its limitations in clayey soils, the standard penetration test provided results that matched well with previous observations due to the high penetration resistance of the highly over-consolidated ground. Down-hole tests and plate load tests were considered less reliable methods due to their limited applicability in this area. This study considers geographical features, laboratory methods, and empirical correlations from in situ tests, and, therefore, provides a well-summarized guideline to evaluate special geotechnical characteristics of the alluvial plain in southeastern Iraq.

Keywords Alluvial plain · Arid regions · Geotechnical design parameters · In situ test · Laboratory test · Southeastern Iraq

Brief: *Geotechnical site characterization for Southeastern Iraq*

✉ Ilhan Chang
ilhan.chang@unsw.edu.au

Jinung Do
jdo@ncsu.edu

Ong Heo
ong1126@nate.com

Yeo-Won Yoon
yoonyw@inha.ac.kr

[1] Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908, USA

[2] Department of Civil Infrastructure Engineering, Inha University, Incheon 22212, Republic of Korea

[3] School of Engineering and Information Technology (SEIT), University of New South Wales (UNSW), Canberra, ACT 2600, Australia

Introduction

Damage to and failures in civil structures are mainly caused by factors such as stress imbalances, seismic events, extreme environmental conditions (e.g., wind, flood, and droughts), and deterioration of the construction (Feld and Carper 1997). Excluding natural disasters, about 60% of structural failures are attributed to errors in the planning or design stage (Love et al. 2013; Sowers 1993), where many historical failures are due to misjudgments of in situ geotechnical engineering parameters (Table 1).

Accurate site characterization is the first step for reliable geotechnical engineering design. Generally, in situ soil properties depend on the geological processes of the soil's origin. The geological origin of arid soils differ from soils in temperate and tropical climates, resulting in distinctive geotechnical characteristics (Chang et al. 2015; Holtz et al. 2011). Typical characteristics of arid soils are as follows: (1) low organic content, (2) intensive precipitation and severe surface erosion

Table 1 Historical construction failures caused by geotechnical design errors

Year	Name	Location	Causes	Consequences	Reference
1300s	Leaning tower of Pisa	Pisa, Italia	Inadequate foundation on alluvial sediments	Building inclined 5.5°	Burland et al. (1998)
1976	Teton Dam	Idaho, USA	Piping or flow through grout curtain	14 deaths; large downstream areas destroyed; costs of 1 billion USD	Sherard (1987)
1998	Tram Tunnel	Den Hague, Netherlands	Leakage due to cost-saving inadequate construction technique	Delays; costs increased from 139 million to 234 million euros	Baars (2005)
2000	Pier 34 Heat nightclub	Philadelphia, Pennsylvania, USA	Deterioration by wetting and drying of the timber piles	41 people plunged into the water; 3 deaths	Bishop (2000)
2009	Light structures	Ankara, Turkey	Heaving due to expansive soil after heavy rain	Structure collapsed	Ozer et al. (2012)
2009	Lotus Riverside Complex	Shanghai, China	Ground instability caused by dumping excavated soil on a nearby river bank	Building collapsed; 1 death	Jianhua (2010)
2009	Minard Hall Façade	Fargo, North Dakota, USA	Soft ground; loss of bearing or a disturbance from adjacent work	Building collapsed	North Dakota State University (2011)

during a limited rainy season (one or fewer weeks per year), (3) preferential drainage due to the main composition of coarse particles (mostly sands), (4) rapid surface evaporation and poor soil moisture retention, and (5) eolian dusts (mostly clay) and mechanical weathering of coarse grains. These specific conditions in arid soils lead to the formation of landforms such as bajadas, pediment and piedmont plains, fans, and playas (West 1995). On the other hand, eroded and transported particles form alluvial clayey deposits with the reduction of driving matters (wind or water flow).

When an arid alluvium is exposed to hydrogeological effects, dehydration of soil moisture induces severe shrinkage and accompanying desiccation cracks on the surface (Tang et al. 2011). Surface desiccation reveals preconsolidation effects on arid soil surfaces (Morris et al. 1992) which leads to difficulties in site characterization (Fredlund et al. 2012; Houston et al. 2001; Livneh et al. 1995; Poulos and Davis 1974). However, detailed wetting and swelling behaviors of most arid soils are still unknown (Houston et al. 2001). In Middle Eastern countries, the plate load test (PLT) is commonly used to evaluate the ground-bearing capacity; however, the reliability of the PLT method is limited to near-surface purposes due to its assumption of uniform geometry underneath (Poulos and Davis 1974). Therefore, accurate and reliable site characterization for arid alluvial soils is an important task for geotechnical engineering. In this study, a series of laboratory and field tests were performed and analyzed for reliable geotechnical characterization of alluvial deposit sites while considering the geological background of southeastern Iraq.

Site characterization

Site of interest

The site of interest in southeastern Iraq (3,673,087.96° N and 464,161.22° E) is located 25 km from Baghdad and is also known as the site of the "Bismayah New City Project" (Fig. 1). The site mainly consists of an alluvial plain deposited by the Tigris and Euphrates rivers. The major ground condition is permanent marsh that is mostly dried near the surface except during occasional annual floods (Flint et al. 2011). In general, the site has a flat topography and a slightly irregular surface. Fookes (1978) classified the geometry of the Middle East region into the following four categories: (1) mountainous areas, (2) large gravel fans surrounding mountains, (3) alluvial plains beyond the fans, and (4) central base level plains. The in situ condition of this study is classified as category 3 (Fookes 1978). Four specific locations (A, B, C, and D) were randomly selected to represent the site of interest (Fig. 1). An in situ sampling and comprehensive laboratory and field tests were performed for each location simultaneously in this study.

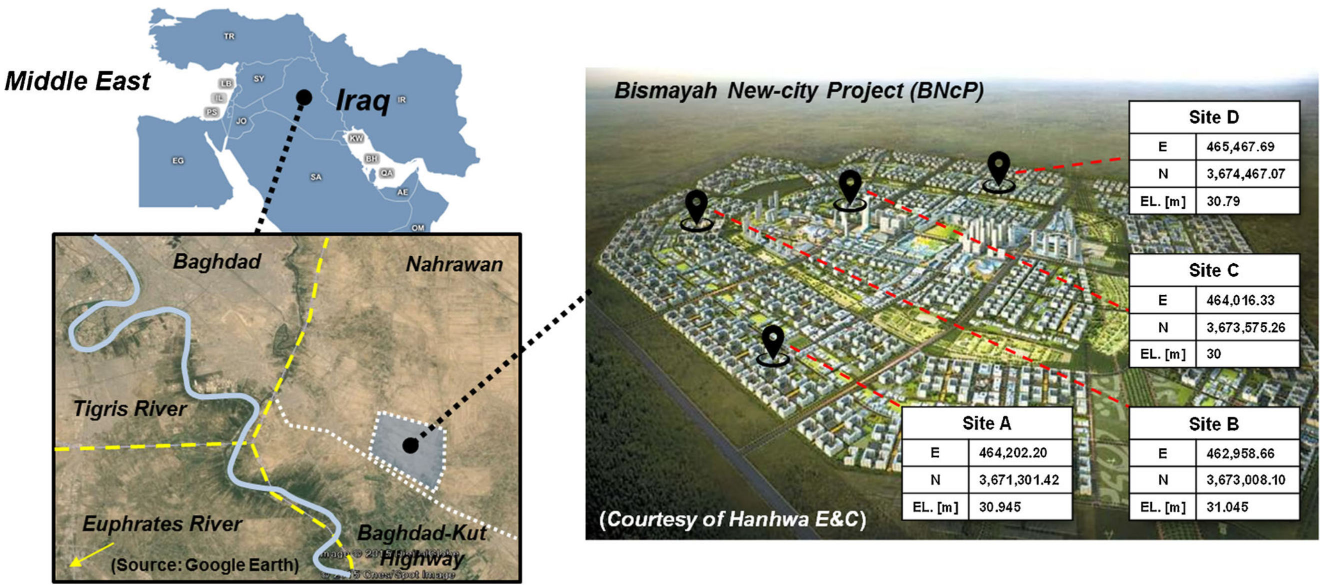


Fig. 1 Test locations (Alluvial plain in southeastern Iraq along the Tigris and Euphrates rivers)

Laboratory programs

Basic soil properties

Undisturbed samples were collected via Shelby Tube and split-spoon sampler depending on the depth in situ. The in situ surface layer (0–2 m depth) mainly consists of silts (*MH* or *ML*), while clayey soil (*CL*) becomes dominant at depths of 4–16 m, and sandy soils (*SP*) become dominant at depths greater than 18 m. The groundwater table is near the surface (at 2 m depth) due to the Tigris and Euphrates rivers, which almost surround the site.

Basic soil properties such as in situ water content, particle size distribution, and Atterberg limits were assessed for each location by referring to ASTM D2216 (ASTM 2010), ASTM D4318 (ASTM 2017), and ASTM D854 (ASTM 2014a), respectively. In situ soils were classified according to the unified soil classification system (USCS). The in situ groundwater table was measured by well-logging according to ASTM D6724 (ASTM 2016). The basic in situ geotechnical properties obtained are summarized in Table 2. In addition, unconsolidated undrained (*UU*) triaxial tests were conducted to determine in situ shear strength parameters at 0–15 m depths following ASTM D2850 (ASTM 2015).

Compressibility characteristics

Laboratory consolidation tests were implemented via laboratory odometer apparatus by following ASTM D2435 (ASTM 2011a). The specimen size was 63.5 mm in diameter and 25.4 mm in height. Overburden pressures of 1, 25, 50, 100, 200, 400, and 800 kPa were subsequently applied via step loading where each step was applied for 24 h to ensure the dissipation of excess pore water pressure. After the completion of the loading, samples were unloaded to 200 kPa and then 50 kPa. Compressibility parameters, such as coefficients of consolidation (C_v), recompression index (C_r), compression index (C_c), and over-consolidation ratio (OCR; σ'_p/σ'_{vo}), were determined by time-dependent stress–deformation relationships obtained from laboratory test results (σ'_{vo}: preconsolidation pressure, σ'_{vo}: effective overburden pressure) (Chang et al. 2011; Taylor 1942). Compressibility parameters obtained by the consolidation tests were compared with those estimated via empirical correlation methods (Hong and Onitsuka 1998; Park and Koumoto 2004; Rendon-Herrero 1983; Terzaghi et al. 1996) using field data such as specific gravity (G_s), liquid limit (*LL*), and in situ void ratio (e_0) or porosity (n_0) as summarized in Table 3. Overall, compression indices obtained via laboratory tests and empirical prediction methods show similar results ($C_c = 0.13$–0.18) within the typical range of low plastic silty soil (*ML*) (Kaufman and Sherman 1964), except the estimations obtained via the model of Terzaghi et al. (1996) ($C_c = 0.30$). This is because the method of Terzaghi et al. (1996) only considers the *LL* of soil, without in situ conditions (e.g., void ratio).

In addition, the laboratory consolidation results (Table 3) show OCR values of 4–5 (2 m), 1–2 (10 m), and 1 (16 m), where OCR decreases with greater depths. The in situ average C_v (5.3 m^2/year) is similar to the typical C_v value of over-consolidated low plastic clays (3.8m^2/year, compared with 110 m^2/year for recompressed clay and 15 m^2/year for undisturbed clay) (Chang and Cho 2010; Lambe and Whitman 1979), which implies the densification effect via surface geological history. The over-consolidation effect is of concern due to the high degree of swelling when re-wetted. Thus, not only the physical compressibility properties

Table 2 Engineering properties obtained by laboratory tests

Site	Depth [−m]	D_{50} [mm]	C_u	C_c	G_s	w [%]	LL [%]	PI [%]	WL [−m]	$USCS$
A	1.5	0.002	NA	NA	2.78	25.69	61	16	2.1	MH
	5.5	0.005	NA	NA	2.73	19.94	38	14		CL
	10.5	0.0025	NA	NA	2.74	29.33	44	18		CL
	15.5	0.0025	NA	NA	2.71	19.14	46	20		ML
	20.25	0.25	2.25	1.23	2.69	27.23	NA	NA		SP
	45.25	0.23	2.5	1.11	2.68	25.04	NA	NA		SP
B	1.5	0.0015	NA	NA	2.76	21.03	34	13	1.9	CL
	4.5	0.002	NA	NA	2.77	17.2	36	14		CL
	7.5	0.0025	NA	NA	2.75	30.17	38	15		CL
	10.5	0.002	NA	NA	2.72	21.65	46	23		CL
	18.25	0.24	2.7	1.20	2.69	26.29	NA	NA		SP
	40.25	0.25	0.75	0.33	2.67	22.74	NA	NA		SP
C	1.5	0.001	NA	NA	2.74	22.31	47	18	2.5	ML
	5.5	0.0025	NA	NA	2.74	20.24	45	20		CL
	10.5	0.0015	NA	NA	2.73	19.87	42	17		CL
	17.25	0.25	2	0.86	2.71	20.95	NA	NA		SP
	27.25	0.22	2.56	1.24	2.67	20.33	NA	NA		SP
	45.25	0.21	2.45	1.09	2.68	22.33	NA	NA		SP
D	1.25	0.001	NA	NA	2.74	21.8	41	14	2.1	ML
	5.75	0.005	NA	NA	2.78	26.97	45	16		ML
	10.25	0.0025	NA	NA	2.76	30.2	46	20		ML
	12.75	0.003	NA	NA	2.71	23.34	43	17		ML
	16.75	0.005	NA	NA	2.68	32.27	35	11		CL
	26.25	0.21	2.4	0.94	2.66	21.21	NA	NA		SP

D_{50}: mean particle size; C_u: coefficient of uniformity; C_c: coefficient of curvature; G_s: specific gravity; w: in situ water content; LL: liquid limit; PI: plastic index, WL: depth of ground water table, NA: not available

but also the ground water table and the neighboring hydrological circumstances must be considered simultaneously for alluvial plains in this region.

Field programs

Hydraulic conductivity

In situ hydraulic conductivities at 5–6 m depths were assessed by following ASTM D6391 (ASTM 2011b). In situ hydraulic conductivity values (in cm/s unit) of 1.01×10^{-5} (location A), 2.31×10^{-6} (location B), 6.90×10^{-6} (location C), and 2.74×10^{-6} (location D) were obtained, which are in accordance with the typical permeability range of clayey soils (Chang and Cho 2010; Lambe and Whitman 1979).

Standard penetration test (SPT)

Standard penetration tests (SPTs) were performed at each location (A, B, C and D) in the field according to ASTM D1586 (ASTM 2011c). The measured standard penetration numbers (N_{measured}) were converted to N_{60} by considering 60% energy efficiency (Skempton 1986) (Fig. 2). N_{60} values were thereafter converted to $(N_1)_{60}$ values by considering both atmospheric pressure (p_a: 101 kPa) and the in situ effective overburden pressure correction factor C_N. Different C_N formulas were compared simultaneously (Liao and Whitman 1986; Peck et al. 1974; Seed et al. 1986). As shown in Fig. 2, N_{60} values are in the range of 13–19 at 2 m depth, decrease slightly to 9–14 down to 5 m depth, and start to increase gradually at 5 m and greater depths. Overall, N_{60} and $(N_1)_{60}$ values show similar trends, although $(N_1)_{60}$ values become higher than N_{60} near the surface, reflecting the high OCR requiring higher penetration resistance. The N_{60} values (15–30) measured at depths of 2–14 m indicate a "very stiff" soil condition, which implies a high OCR condition at shallow depths, while $(N_1)_{60}$ values at 16–30 m depths represent "medium" to "dense" soil conditions (Das 2016; Karol 1960; Peck et al. 1974).

Due to the impact of the hammer, the SPT requires one of keen understanding to apply the empirical equations for cohesive soils, especially deposited alluvial plains with high OCR caused by geological actions. The correction factors, however, still do not account for many uncertainties (e.g., cohesive or granular soils, and normally consolidated or over-consolidated soils), and the representative ranges are too broad and approximate to specify the soils in detail. Thus, more precise and continuous site measurements via cone penetration tests were conducted.

Cone penetration test (CPT)

In situ cone penetration tests (CPTs) were performed for 0–20 m depths at the same locations where the SPTs were implemented following the ordinary method measuring both cone tip resistance (q_t) and friction resistance (f_s) simultaneously during penetration (Lunne et al. 1997). Measured q_t and f_s data are plotted in Fig. 3. q_t increases from 0 to 1 m depth, and then decreases with depth down to 10 m. For depths greater than 10 m, q_t increases due to the increase of cohesionless deposits in ground. The f_s profile shows a similar trend to that of q_t. CPT measurements obtained at 0–2 m, 6–8 m, and 18–20 m depths were analyzed and classified by the normalized CPT soil behavior type (SBT_n) suggested by Robertson (2010), as shown in Fig. 4. SBT_n is generally used to classify the soil using CPT measurements considering σ_{vo}, soil density, stress history, and sensitivity. Most in situ soils (0–20 m) are categorized as over-consolidated soils, especially in the near-surface layer (0–2 m), which is classified as "very stiff sand to clayey sand" or "very stiff fine grained" soil. Although the SPT method is effective for evaluating the in situ soil type and strength profile, it has limitations in indicating the in situ stress history. However, with SBT_n

Table 3 Compressibility properties obtained by laboratory tests and empirical correlations

Location	Depth [−m]	e_o	C_v [m^2/year]	C_r	C_c					OCR
					Lab	TZ	RH	HO	PK	
A	1.5	0.55	3.74	0.024	0.13	0.46	0.12	0.2	0.16	4.28
	5.5	0.58	4.67	0.02	0.24	0.25	0.13	0.13	0.17	1.22
	15.5	0.87	4.06	0.028	0.25	0.32	0.19	0.16	0.27	1.12
B	1.5	0.75	3.89	0.016	0.11	0.22	0.16	0.12	0.23	4.48
	7.5	0.55	4.67	0.03	0.09	0.25	0.12	0.13	0.16	1.22
	10.5	0.53	6.67	0.006	0.12	0.32	0.12	0.16	0.15	1.55
C	5.5	0.53	3.11	0.017	0.13	0.32	0.12	0.16	0.15	2.45
	10.5	0.55	4.66	0.026	0.16	0.29	0.12	0.15	0.16	1.55
D	5.8	0.57	3.11	0.027	0.16	0.32	0.12	0.16	0.17	3.42
	12.8	0.61	9.33	0.029	0.13	0.3	0.14	0.15	0.18	0.61
	16.8	0.53	10.38	0.018	0.18	0.23	0.12	0.12	0.15	1.21
Average	–	–	5.3	0.022	0.16	0.3	0.13	0.15	0.18	–

Lab: $\Delta e/\Delta \log(\sigma'_{vo})$

TZ: $0.009(LL - 10)$ (Terzaghi et al. 1996)

RH: $0.141 G_s^{1.2} \{(1+e_o)/G_s\}^{2.38}$ (Rendon-Herrero 1983)

HO: $0.332 \log LL - 0.390$ (Hong and Onitsuka 1998)

PK: $n_o/(371.747–4.275 n_o)$ (Park and Koumoto 2004)

taken into consideration, the CPT method becomes more effective in the current site evaluation as well as in identifying the history in the field.

Down-hole test

Down-hole tests (DHTs) were performed with signal receivers (three-dimensionally aligned geophones: ABEM Terraloc Pro, Guideline Geo) cased with galvanized pipe (7.62 cm in diameter) and placed into drilled boreholes down to 30 m depth. A standard SPT hammer (76 cm diameter; 63.5 kg weight) was used to generate seismic impact from the surface 3 m away from the borehole entrance. A band-pass filter ($1\ \text{kHz} \leq f \leq 50\ \text{kHz}$) was used to remove unwanted noises and obtain clear first arrival signals. The shear wave velocity ($V_s = \Delta d/\Delta t$) was evaluated by measuring first arrival time (Δt) and path of signal (Δd). Then, the stress-corrected shear wave velocity (V_{s1}) was determined by $V_s(p_a/\sigma'_{vo})^m$ (Hoar and Stokoe 1978) where the exponent m is set as 0.25 for clean sands and 0.5 for cohesive soil (Yamada et al. 2008). Both in situ V_s and V_{s1} profiles are plotted in Fig. 5. Unlike the SPT (Fig. 2) and CPT (Fig. 3) results, most DHT results show gradual increases from the surface, and V_s and V_{s1} show a similar trend. Values of V_{s1} characterize the in situ profile as stiff clay (65–140 m/s at 2 m depth), dense sand with gravel (200–410 m/s at 4–12 m depths), residual soil (300–600 m/s at 12–20 m depths), and moderately to highly weathered rock (760–3000 m/s at depths below 20 m) (Hunt and Hunt 2005; Kavazanjian Jr et al. 1997), which is inaccurate compared to evaluation results from laboratory, SPT, and CPT approaches.

Since shear wave velocities are small-strain ($< 10^{-4}$%) parameters, DHT results are not compatible with large-strain methods such as SPT and CPT. Fundamentally, DHT is appropriate when the ground stiffness (or density or strength) gradually increases with depth (homogeneous) to avoid uncertain refraction concerns along interfaces between layers with different impedances according to ASTM D7400 (ASTM 2014b). In this site, the stiffness mainly decreases from 0 to 4 m depths (Fig. 2 and Fig. 3) due to high OCR (Fig. 4) near the surface. However, the DHT was inappropriate for identifying the initial stiffness reduction (Fig. 5) as well as aging, cementation, and OC effects in situ (Schneider et al. 1999; Vucetic and Dobry 1991). Thus, it can be concluded that the application of DHTs is not suitable for alluvial plains in southeastern Iraq.

Plate load test (PLT)

Plate load tests (PLTs) were performed at 0.5 m depth with a plate diameter (B) of 0.3 m to evaluate the ground-bearing capacity values of each location (Fig. 6). The stress–deformation relationships show a linear trend up to 500 kPa applied stress where the modulus of subgrade reaction (k_1) classifies the subsurface layer as hard clay ($k_1 > 50$ MN/m^3) for all

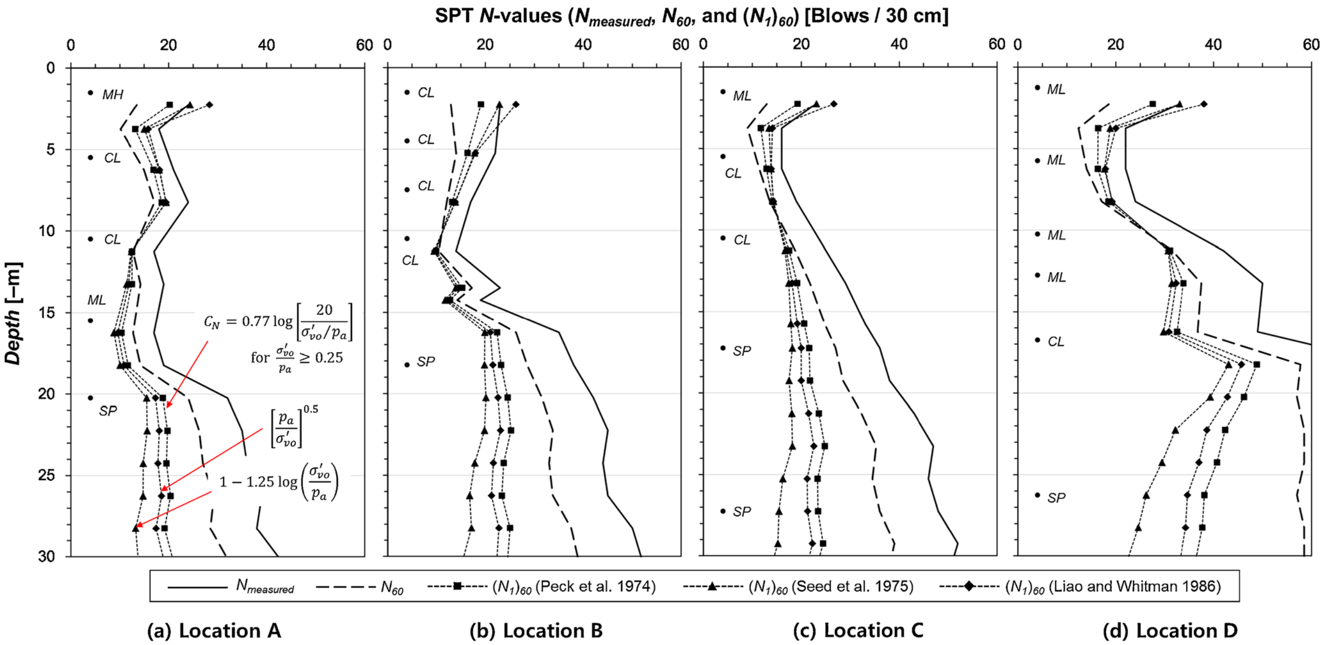


Fig. 2 Standard penetration test results (N_{measured}, N_{60}, and $[N_1]_{60}$) with laboratory soil classification results for (**a**) site A; (**b**) site B; (**c**) site C; and (**d**) site D from Fig. 1

locations (Das 2016). An in situ bearing capacity was evaluated via PLTs to be 700 kPa on average (Table 5), which is higher than evaluations from CPTs (200 kPa) and SPTs (360 kPa) using the equations in Table 4. Since the applicability of the PLT method is restricted to shallow depths (1.5B–2.0B, where B is the width or diameter of loading plate) from the position of loading (Poulos and Davis 1974), PLT measurements only represent the near-surface high OCR layer for this site, giving rise to concerns about overestimating the surface bearing capacity values. Thus, PLT should be considered together with other approaches (e.g., SPT, CPT, etc.) to avoid design errors, especially for shallow foundations in this region.

Geotechnical engineering design parameters for alluvial plain deposits in southeastern Iraq

In situ density

The dry unit weight (γ_d) of soil is an effective geotechnical engineering parameter to identify in situ density characteristics (e.g., void ratio). Figure 7 shows γ_d values derived from laboratory tests and in situ CPT measurements. As G_s, w, and γ_w were obtained from undisturbed samples, γ_d values obtained from laboratory tests are regarded as references ($\gamma_d = \gamma_t/[1 + w]$). CPT measurements can be used to estimate in situ γ_d (Ku et al. 2013; Robertson 2009) (Table 4). Figure 7 shows

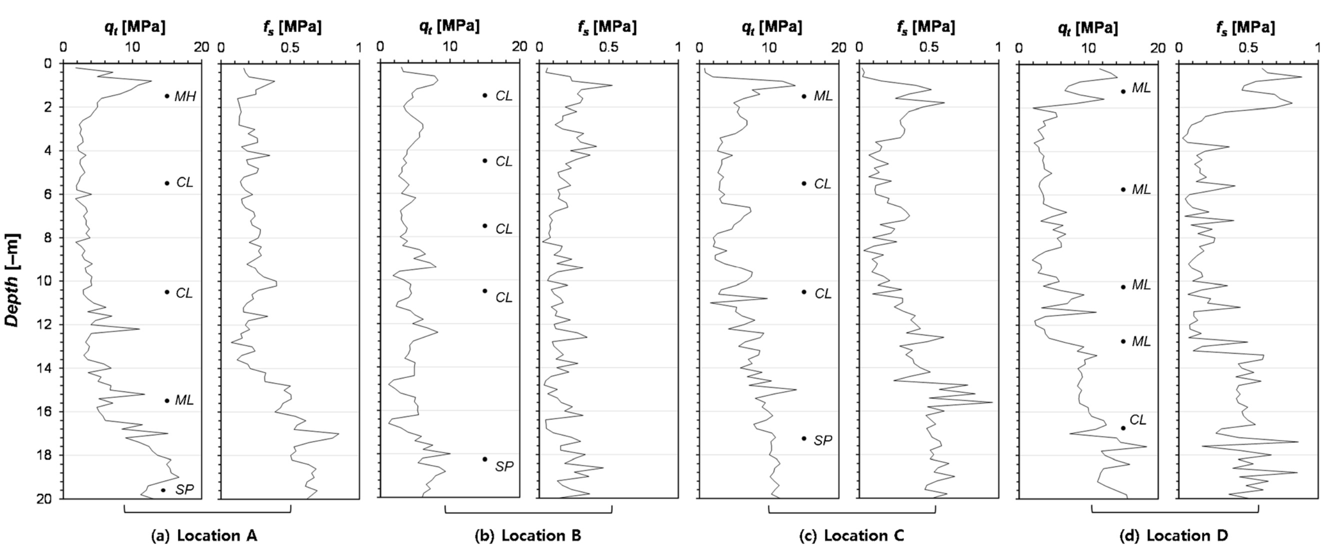


Fig. 3 Cone penetration test results (q_t and f_s) with laboratory soil classification results for (**a**) site A; (**b**) site B; (**c**) site C; and (**d**) site D from Fig. 1

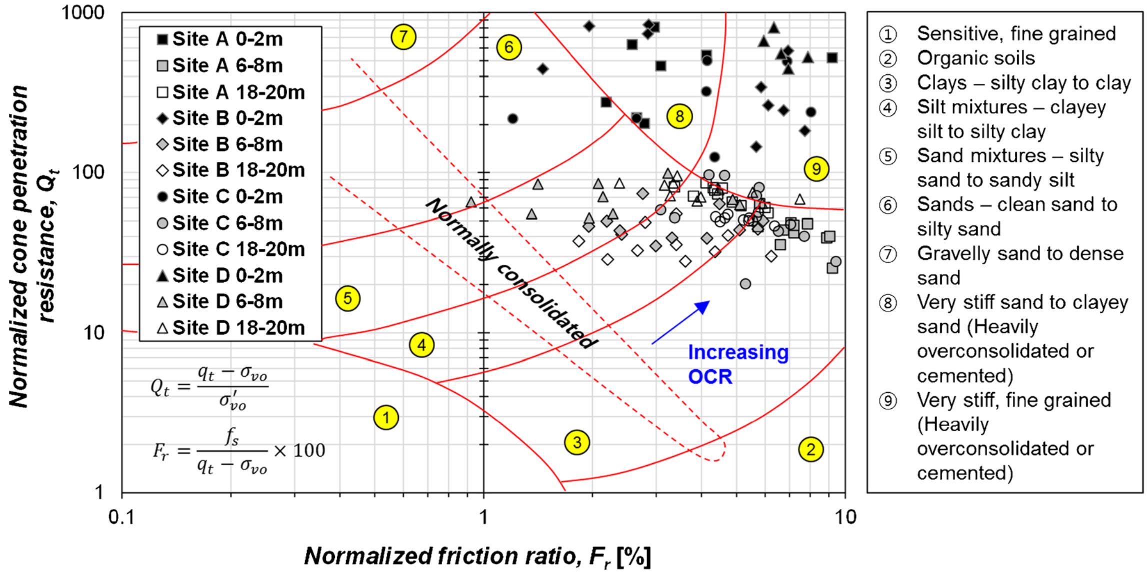


Fig. 4 Normalized CPT soil behavior type (SBT_n) with in situ values categorized at depths of 0–2 m (black solid symbol), 6–8 m (gray solid symbol), and 18–20 m (open symbol) (Robertson 2009)

that γ_d values considering both q_t and f_s CPT measurements (Robertson 2009) become similar to γ_d data from laboratory tests, while only considering f_s (Ku et al. 2013) seems to overestimate (by approximately 10%) the γ_d due to the overconsolidated condition that results in high f_s measurements on site. Thus, considering both q_t and f_s can provide close-to-true in situ density when CPTs are used in southeastern Iraq.

Elastic and shear modulus

In situ elastic modulus (E) and shear modulus (G) at small-strain are obtained from DHT measurements. G values can be derived from in situ V_s and density data, while E via DHT (E_{DHT}) can be obtained using the theoretical relationship between E and G ($E = 2G[1 + \nu]$). Generally, Poisson's ratios (ν) of soils are assumed to be 0.5 for the clayey soils and 0.33 for the sandy soils due to the difficulties in exact measurement in the field (Bishop and Hight 1977; Bowles 1996). Theoretical and empirical equations to obtain E values from field measurements are summarized in Table 4, and evaluation results on in situ E values are plotted in Fig. 8.

Both E_{SPT} and E_{CPT} values for the upper clay layer (0–10 m depths) are in accordance with the typical E range of stiff clays (Bowles 1996; Kulhawy and Mayne 1990). Since E_{DHT} represents the small-strain (10^{-4}%) modulus, there are differences between E_{DHT} and elastic modulus values from large-strain

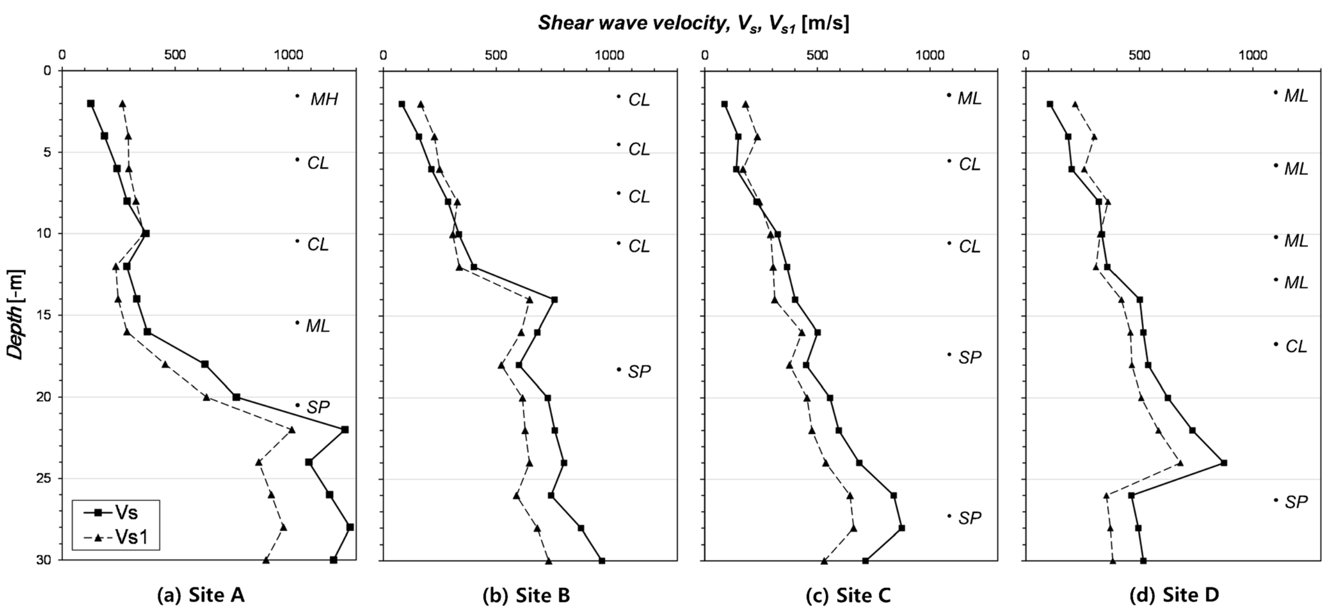


Fig. 5 Down-hole test results (V_s) with laboratory soil classification results for (**a**) site A; (**b**) site B; (**c**) site C; and (**d**) site D from Fig. 1

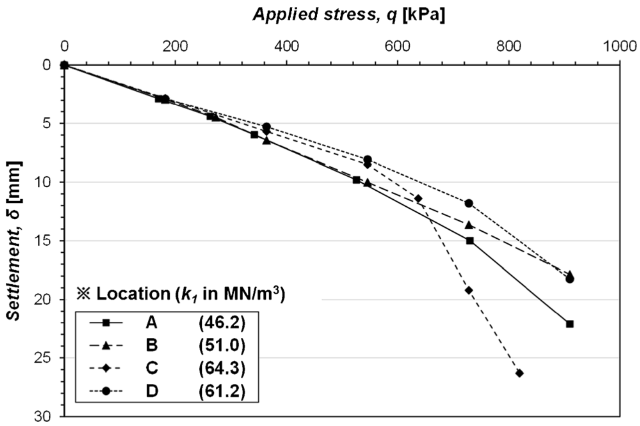


Fig. 6 Results of plate load tests with the modulus of subgrade reaction

measurements (e.g., SPT, CPT, and PLT) (Seed et al. 1986). However, E_{SPT} gradually increases with greater depths, while E_{CPT} exceeds E_{SPT} for depths greater than 10 m. In contrast, E_{DHT} shows the highest values as a result of small-strain measurements, which require correction factors to be applied in the design. E_{PLT} results show unreliable data (5.06–6.08 MPa) due to the limited applicability (in depth) of PLTs.

Over-consolidation ratios

The results of both laboratory and field tests indicate that the main geotechnical feature of alluvial plains in southeastern Iraq is that they are highly over-consolidated near the surface. OCR values obtained from laboratory, SPT, and CPT measurements (via equations in Table 4) are shown in Fig. 9. In general, SPT is regarded to be inappropriate for clayey soils due to soil fabric disturbance during penetration impacts, while the OCR values from SPT measurements show a similar trend to the OCR values from CPTs (Fig. 9). Meanwhile, OCR values from laboratory tests are always lower than OCR values from field measurements, which seems to be an effect of stress release or sample disturbance (Ladd 1991; Lunne et al. 2006). In fact, undisturbed sampling is essential for reliable evaluation of OCR and other stress–strain relationships in laboratory testing. Thus, well-performed field tests can be more promising than laboratory tests for OCR evaluation of alluvial plains in southeastern Iraq.

Shear strength parameters

Effective friction angle (ϕ') and undrained shear strength (s_u) are important strength parameters in geotechnical engineering. Various correlations have been proposed to obtain ϕ' and s_u from in situ measurements such as those obtained from SPTs and CPTs as listed in Table 4. Figure 10 provides s_u and ϕ' variations with depth from laboratory and field data. As shown in Fig. 10a, s_u obtained by laboratory *UU* triaxial tests ($s_{u(Lab)}$) shows a wide variation (10–420 kPa), while $s_{u(CPT)}$ and $s_{u(SPT)}$ show similar trends within the s_u range of 100–300 kPa. Both $s_{u(CPT)}$ and $s_{u(SPT)}$ are in accordance with the general s_u values of stiff or hard clays (Reese and Welch 1975). For ϕ' (Fig. 10b), both ϕ'_{SPT} and ϕ'_{CPT} show high values near the surface, while the values decrease with greater depths. The overall ϕ' variation is similar to the OCR variation (Fig. 9). As both ϕ' and OCR are physical parameters related to the stress state and accompanying inter-particle density, it becomes reasonable to have similar OCR and ϕ' distributions for the same geometry. However, for sandy soil layers (depths of 10 m and greater), ϕ'_{SPT} shows a wider variation (33–48°), while the variation of ϕ'_{CPT} becomes narrower (38–42°) than ϕ'_{SPT}. Moreover, the ϕ'_{CPT} values matched the typical ϕ' (36–41°) of dense sand (Peck et al. 1974). Despite the suitability of SPT for sandy soils, the correlations suggest that CPT is more reliable than SPT.

In general, the impact energy of SPT causes the structural collapse of the clay's fabric (e.g., flocculated → dispersed structure), which leads to changes in the undrained shear strength. The correlation results of SPTs in Fig. 10, however, show reasonable values with that of CPTs. This implies that firm fabrics induced by high OCR better endure the impact energy of SPT preventing the structural collapse of the soils than moderately consolidated clay. Some research shows that the impact energy of SPT has less effect on high OCR soils (Holtz et al. 2011; Mayne and Kemper 1988). Nevertheless, the application of SPT in southeastern Iraq requires profound consideration and technical understanding due to the special characteristics of alluvial plains in this region. Overall, the shear strength parameters obtained by laboratory tests show less reliability due to the sample disturbance effect of highly over-consolidated soil, while in situ CPT measurements seem to be appropriate to evaluate shear strength parameters (Fig. 10) for this site. Despite the lower applicability of SPTs to clayey soils, SPT correlations follow similar trends to CPT correlations.

Discussion

Overview of the geological timeline of alluvial plains in southeastern Iraq

The series of laboratory and field tests from this study verify that the in situ surface layer is highly over-consolidated. Generally, geological processes such as loading and unloading (e.g., as a result of glaciation, sedimentation, and uplift), fluctuations in groundwater level, desiccation due to repeated drying and wetting, cyclic freezing and thawing, and chemical bonding and cementation are known to induce over-consolidation behaviors in situ (Wair et al. 2012). Most alluvial plains in southeastern Iraq were formed during the early

Table 4 Empirical correlations for geotechnical design parameters

Parameters	Source	Equation	Note	Reference
OCR	Lab	σ'_p/σ'_{vo}		Casagrande (1936)
		$\frac{137.924}{\sigma'_{vo}}-\frac{137.924}{\sigma'_{vo}}\left(\frac{e_o}{e_L}\right)-0.179$	e_L: void ratio at liquid limit ($LL/100\times G_s$)	Solanki and Desai (2008)
	SPT	$0.193\left(\frac{N_{60}}{\sigma'_{vo}}\right)^{0.689}$	σ'_{vo} in MPa	Mayne and Kemper (1988)
	CPT	$\frac{1}{\sigma'_{vo}}\times 0.33(q_t-\sigma_{vo})^{m'}\left(\frac{Pa}{100}\right)^{1-m'}$	$m'=1-\frac{0.28}{1+(I_c/2.65)^{25}}$ $I_c=\sqrt{(3.47-\log Q_t)^2+(1.22+\log F_r)^2}$	Ku et al. (2013); Robertson (2009)
γ_t [kPa]	Lab	$\frac{(G_s+wG_s)\gamma_w}{1+wG_s}$	γ_w: unit weigth of water in kN/m^3	Das (2016)
	CPT	$\gamma_w\times\left(0.27\log R_f+0.36\log\frac{q_t}{p_a}+1.236\right)$	R_f: friction ratio ($f_s/q_t\times 100$) in percent	Robertson (2009)
		$26-\frac{14}{1+[0.5\log(f_s+1)]^2}$	f_s: friction resistance in kPa	Ku et al. (2013)
s_u [kPa]	SPT	$0.29p_a(N_{60})^{0.72}$	p_a in kPa	Hara et al. (1974)
	CPT	$\frac{q_t-\sigma_{vo}}{N_k}$	N_k: cone factor in accordance with PI and sensitivity. 25 for OC clay	Lunne et al. (1997)
ϕ' [°]	SPT	$\tan^{-1}\left[\frac{N_{60}}{12.2+20.3\left(\frac{\sigma'_{vo}}{Pa}\right)}\right]^{0.34}$		Kulhawy and Mayne (1990)
	CPT	$\tan^{-1}\left[0.38+0.27\log\frac{q_t}{\sigma'_{vo}}\right]$	For *ML* and *SP-SM*	Ricceri et al. (2002)
E [MPa]	SPT	$40+0.963N_{60}$		Bowles (1996)
	CPT	$0.015\left[10^{(0.55I_c+1.68)}\right](q_t-\sigma_{vo})$	q_t and σ_{vo} in MPa	Robertson (2009)
	DHT	$\rho V_s^2(1+\nu)$	Initial modulus, ρ: mass density of the soil	Lambe and Whitman (1979)
	PLT	$\frac{P(1-\nu^2)}{2r\delta}$	P: applied load, r: plate radius, δ: settlement at P	Bowles (1996)
Bearing capacity [kPa]	PLT		Yielding point	
	SPT	$14.44N_{\text{measured}}$	$N_{\text{measured}}=25$	
	CPT	$5.14s_u$; $s_u=(q_t-\sigma_{vo})/N_k$	$q_t=5000$ kPa, $N_t=25$ for OC clay	

Pleistocene epoch of the Quaternary period in the Cenozoic era (Jassim and Goff 2006; USGS 2010). During the Pleistocene phase, the alluvial plains were underneath massive glaciers and experienced significant stress release due to the thaw of the glaciers after the last glacial epoch (University of Wisconsin–Extension et al. 2006). The removal of the massive overburden glacier pressure is indicated as the main cause of overall over-consolidation effects in situ. Moreover, desiccation is another factor enhancing OCR values of near-surface marshes in Iraq (Hussain and Grabe 2009). The high concentration of SO_4^{-2} in in situ groundwater (885–1770 mg/l) compared to ordinary drinking water (250 mg/*l*) may be evidence of severe desiccation during the past geological timeline (Lamers et al. 1998; United States Environmental Protection Agency 2008).

Engineering characteristics of soils in southeastern Iraq

The geotechnical characteristics of soils in southeastern Iraq obtained from this study are summarized in Table 5. In general, the ground consists of clayey soils in the upper layer (0–15 m) and sandy soils in the lower layer (15 m and deeper). The site characterization can be performed by various approaches, while the CPT approach provides the most sophisticated descriptions, including the stress history and sensitivity of the soils. The in situ density can be found through

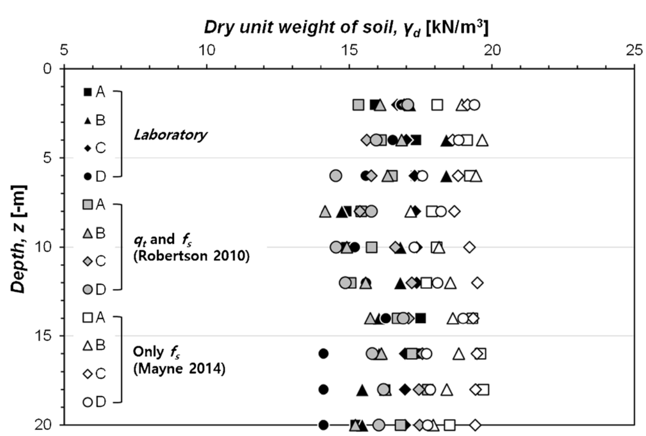
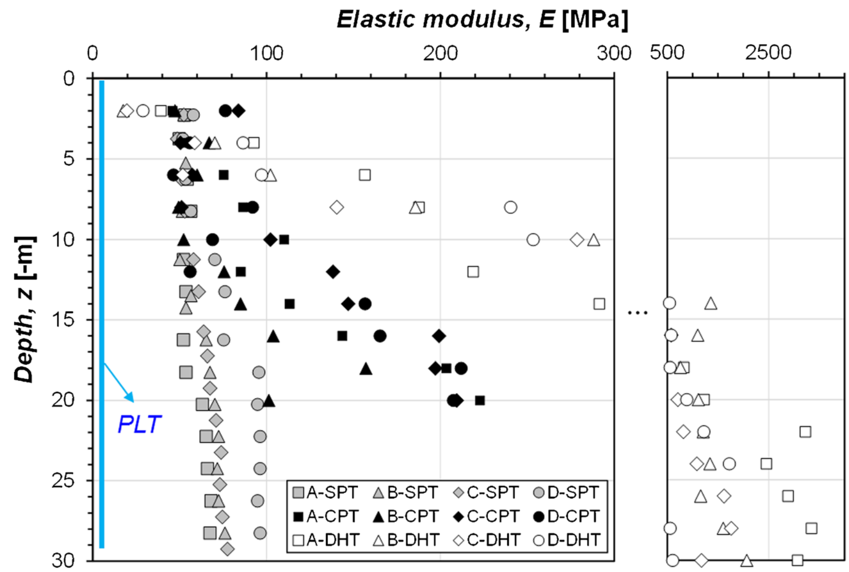


Fig. 7 Comparisons of soil unit weight obtained using laboratory and CPT correlations

Fig. 8 Comparison of elastic modulus obtained using SPT, CPT, DHT, and PLT correlations



laboratory tests and CPTs, but the laboratory tests require delicate sampling technologies and low disturbance. The s_u values increase with depth due to increasing confinement, while the ϕ' values are highest near the ground surface due to high OCR. Thus, s_u does not adequately indicate the overconsolidated condition at the shallow depth in southeastern Iraq, so it is recommendable to investigate to at least a 15 m depth to avoid the overestimation of the strength parameters. The E_{SPT} values show narrow ranges due to the simplification of the empirical correlation, however E_{CPT} values provide more realistic variations. For the application of E_{DHT} values, the strain dependent stiffness reduction (e.g., G/G_{max} curve) has to be considered. Meanwhile, the PLT method overestimates the bearing capacity due to the limited zone of influence.

Data from Mohammed and Abdulrassol (2017), a study conducted in Baghdad, Iraq, supports the evaluations in this study (Table 6) in terms of the soil classification and dry density, while the shear strength, friction angle, and elastic modulus calculated by V_s show some discrepancies. The shear strengths obtained by the laboratory testing seem similar with what Mohammed and Abdulrassol (2017) proposed, but these values differ from those obtained by empirical correlations (e.g., SPT, CPT). In a similar manner, data from the deltaic alluvial plains of Bangladesh presents similar

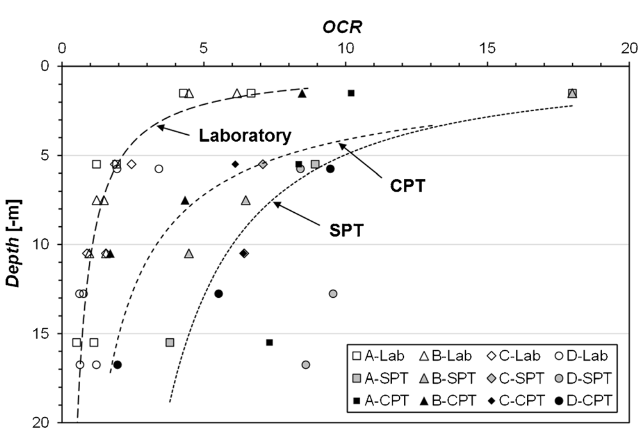


Fig. 9 Comparison of over-consolidation ratios obtained using laboratory results, SPT, and CPT correlations

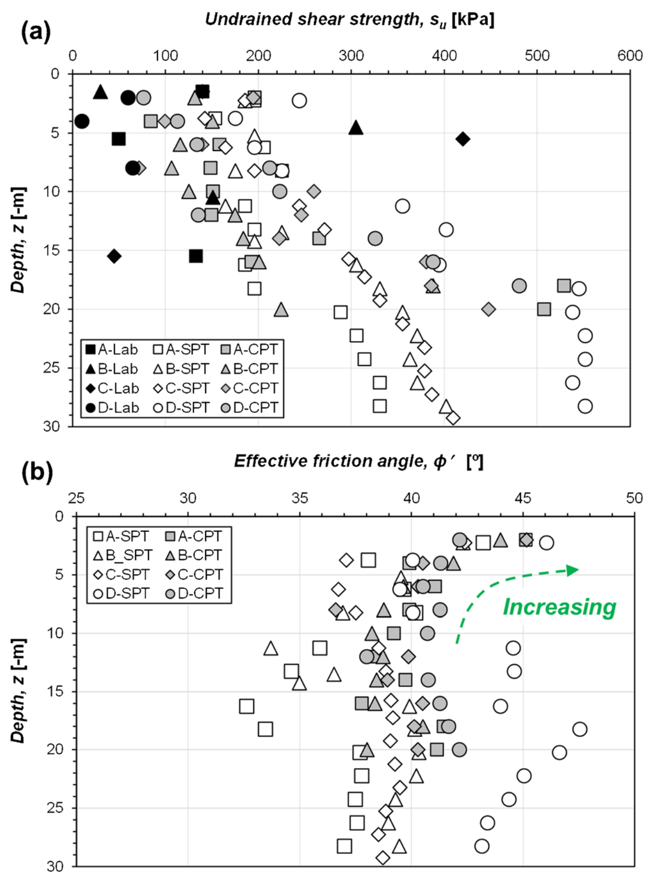


Fig. 10 Comparison of (**a**) soil cohesion and (**b**) effective friction angle obtained using laboratory results, SPT, and CPT correlations

Table 5 Overall engineering properties obtained by diverse tests

Methods	Classification			OCR			γ_d [kN/m^3]			s_u [kPa]			ϕ' [°]			E [MPa]			Bearing capacity [kPa]
Depth [m]	0–2	6–8	18–20	0–2	6–8	18–20	0–2	6–8	18–20	0–2	6–8	18–20	0–2	6–8	18–20	0–2	6–8	18–20	1–1.5
Laboratory	Silt and clay	Clay	Sand	4–7	2	1	16–17	15–17	14–17	15–200	50–400	NA	NA	NA	NA	NA	NA	NA	NA
SPT	Very stiff	Very stiff	Medium to dense	18	4–8	3–8	NA	NA	NA	180–240	160–220	200–550	42–46	36–40	32–48	50	50	50–100	360
CPT	"Very stiff sand to clayey sand" and "very stiff fine grained"	"Silty clay to clay," "clayey silt to silty clay," and "silty sand to sandy silt"		8–10	4–6	2–4	15–19	14–19	16–20	140–200	80–150	220–240	42–45	36–41	38–42	40–80	50–110	200–220	200
DHT	"Stiff clay" to "dense sand and gravel"		"Residual soil" to "moderately-highly weathered rock"	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	20–40	140–300	500–1500	NA
PLT	Hard clay	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	5–6	NA	NA	700

compressibility but lower levels of shear strength as studied by Mollah (1993). As Mollah (1993) proved, Bangladesh plains contain organic silt–peat mixtures erratically, but frequently. In addition, Chung et al. (2002) showed alluvial plains in Busan, Korea to be mostly identified as under and normally consolidated deposits. As addressed by Holtz et al. (2011) and Mollah (1993), shrinkage and swelling can be found in the soils of alluvial plains, and high-intensity rainfall in desert regions often induces the rapid changes of the land by carrying large amounts of sediment by surface discharge. However, those phenomena are often dismissed due to the rareness of the events and lack of high-quality data.

As observed in the literature reviews, even same alluvial plains have unique geotechnical characteristics according to their geological situations, such as an aridity with desiccation; a tropicality with organic, soft ground under-consolidation; etc. Therefore, it is concluded that there is no one universal behavior and characteristic of the soils in alluvial plains, and appropriate site investigations are required to provide a comprehensive understanding of the alluvial plain.

Recommendations for determination of geotechnical design parameters in southeastern Iraq

Using data from Table 5 and previous literatures, the applicability of the diverse testing methods used in this study in southeastern Iraq were evaluated from the geotechnical point of view (Table 7). Laboratory testing is generally regarded as the most reliable source of evaluations. However, this paper addresses the possibility of a disturbance on the sampling and downsizing of the geotechnical parameters, as the soils in this area are heavily over-consolidated and susceptible to swelling after stress release. Therefore, comparing the lab results with empirical correlations would be a reasonable approach to verify the quality of the results. SPTs are reasonably suitable for coarse-grained soils, but not reliable for fine-grained soils because the impact energy disturbs the original fabric of the clayey soils. However, SPT correlations provided reasonable values compared with the other investigated methods due to the high penetration resistance of the highly over-consolidated soils. Nevertheless, using SPTs on clayey soils requires careful consideration and interpretation. DHTs are essential for seismic design, and can assess large domains, however the results are too dispersed and unreliable and require additional factors (e.g., aging, cementation, and reduction factors for different strain levels). PLTs are uniquely suited to evaluating in situ stress–strain relationships, but their applicability is too narrow, and they are restricted to shallow surfaces. On the basis of the overall results, the CPT method appears to be the most robust way for determining geotechnical engineering properties in southeastern Iraq; it yields the most reliable parameters at an affordable cost. However, the CPT method also has some limitations, such as poor applicability in gravely

Table 6 Geotechnical properties in alluvial plains

Reference	Location	Depth [m]	USCS	γ_d [kN/m^3]	γ_t [kN/m^3]	C_c	OCR	s_u [kPa]	ϕ' [°]	V_s [m/s]	E [MPa]
This study*	Baghdad, Iraq	0–2	ML/CL	15–19	–	0.19	10–12	112–213	42–46	100	37–57
		6–8	CL	14–16	–	0.17	3–5	97–257	36–40	250	80–153
		18–20	SP	14–19	–	0.16	2–4	210–395	35–45	620	250–540
Mohammed and Abdulrasool (2017)	Baghdad, Iraq	0–10	CL	17.4	–	–	–	180	0	298	538
		10–15	CL	17.1	–	–	–	68	16	428	995
		15–20	SM	17.0	–	–	–	0	34	507	1389
Mollah (1993)	Bangladesh plain, Bangladesh	0–5	ML/CL	–	–	0.2–0.27	–	0–40	5–15	–	–
		5–10	ML/CL	–	–	0.2–0.27	–	0–30	8–18	–	–
Chung et al. (2002)	Busan, South Korea	0–20	CL/CH	–	15–17	0.5–1.5	0.5–2	0–20	–	–	–
		20–35	CL/CH	–	16–18	0.4–0.7	0.3–1.2	20–30	–	–	–

*Averaged lower values - averaged upper values

soils and limited penetration depth. Thus, a comprehensive analysis is required to determine optimal design parameters, therefore, cross-checking with the laboratory testing results would improve the reliability of the parameters.

Conclusions

Various laboratory and field tests were conducted to evaluate the geotechnical properties of the alluvial plain in southeastern Iraq. Theoretical and empirical equations were used to estimate the geotechnical design parameters, and literature reviews were conducted to contextualize the geotechnical conditions. Geological evidence indicates that the highly over-consolidated state was caused by overburden stress removal through glaciation and desiccation. Because of the highly over-consolidated condition, sample disturbance by swelling occurs and should be carefully assessed when laboratory tests are conducted. In practice, in situ tests should also be conducted with careful understanding of the geological characteristics in southeastern Iraq. CPT provides the best correlations with geotechnical design parameters and incurs only moderate testing costs for in situ density, strength and stiffness parameters, and stress history. For strength and stiffness evaluations, SPT results correlated well with previous observations despite the limited applicability of SPT in clayey soils, as the high penetration resistance of the highly over-consolidated ground allows well-matched SPT correlations. Although SPT is less costly than other tests, proper understanding of the measured and correlated data is still required for them to be deployed successfully. DHT and PLT do not allow explicit interpretation because of their restricted application: the correlated values obtained are less reliable than the results from other methods.

Table 7 Characteristics of testing methods based on geotechnical values for southeastern Iraq

Tests	Remarks	Limitations	Disturbance	Design criteria[a]							Cost per 30 m (USD)
				Classification	γ_t	s_u	ϕ	E	OCR	C_c	
Lab	Soil conditions can be altered	Affected by sample disturbance	High	A	A	B	B	B	B	A	2,790[b,c]
SPT	Very good method for sandy soils	Unreliable in soft clayey soils. Disturbed sample obtained	Moderate	B	C	A	A	A	C	C	1200–2400[d]
CPT	Measures tip and friction simultaneously. Minimum disturbance	Gravel causes problems	Low	A	B	A	A	A	B	C	600–900[d]
DHT	Measures larger volume than small sample	Results too approximate. Affected by hole disturbance and geology	Moderate	C	C	B	C	C	–	–	960–1050[b]
PLT	Good method for uniform and normally consolidated soils	Measured value represents only shallow depths	Low	–	–	–	–	C	–	–	150–220[b,e]

[a] Based on the analyzed results (Bowles 1996; Holtz et al. 2011); A = good, B = middle, C = bad

[b] Estimated cost by Aju Geotec Co., Ltd.

[c] Assumed one test set (soil classification \$90 + consolidation \$340 + triaxial test \$500) per 10 m

[d] Ku et al. (2013)

[e] Lower value for "heavy machinery provided by client"; upper value for "heavy machinery included"

In this study, design parameters evaluated from different alluvial plains were also scrutinized, and the results show that geographical occurrences influence the behavior of the soils in alluvial plains such as desiccation, the presence of organic matter, under-consolidation, etc. This implies it is beneficial for the reconnaissance to be conducted with an understanding of the geological backgrounds. Overall, comprehensive analysis with diverse approaches such as geographical analysis, laboratory tests, and empirical correlations by in situ evaluations should be performed simultaneously to assimilate the distinct geotechnical characteristics of the alluvial plain. This study can be a useful reference and guideline to evaluate in situ geotechnical characteristics in alluvial plains, not only in southeastern Iraq, but also other plains.

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