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On the use of constrained modulus for soil settlement analysis

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Abstract. Soil settlement is a key parameter in engineering design of geotechnical structures. Two approaches have been used for the characterization of soil behaviour under one dimensional compression: the linear - nonlinear, the traditional approach, and the modulus-based approach, the constrained modulus. The constrained modulus approach requires the knowledge of effective stress (σ'), stress exponent (d), and soil modulus number (m). In the current study, the constrained modulus approach was adopted in the settlement analysis. Compressibility data of 130 fine and mixed soils with a variety of composition and mineralogy was used in the analysis. In addition, three common clays, bentonite, sepiolite and attapulgite, was experimentally tested using one-dimensional oedometer compression test and the results were included in the analysis. The analysis shows that the approach of constrained modulus can be effectively used to calculate the settlement of fine and intermediate mixed soils, and the stress exponent (d) varies from 0 for clayey soils to 0.3–0.6 for intermediate silty and clayey sand soils depending on the soil plasticity and particle size distribution. Also, there is a simple relationship with $R^2 = 0.83$ between the soil modulus number, m, and the liquid limit, IL. in that the higher the liquid limit, IL. the lower the soil modulus number, m.

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1. Introduction

Loads of engineering earth structures are transferred down by foundations to underlaying soils and distributed as a stress over the affected layers. Generally, stresses compress soil particles quickly at a small rate without any moisture loose at immediate stage. With a stress increase, a reduction in soil volume occurs due to the expulsion of water from interconnected voids, quickly and insignificant in coarse soils, but slowly and significant in fine-grained soils depending on the soil thickness and the permeability. At the end of the process, that is, when dissipation of excess pore water pressure is all completed, some compression (so-called secondary compression) takes place due to the plastic status of soil mass fabrics. In design, it is necessary to assess the suitability of a soil against both excessive settlement failure and allowable bearing capacity. While stability issues of structures in soil mechanics can often be defined satisfactorily, it is still difficult to estimate settlements and deformations accurately in advance [1]. In spite of the growth of knowledge in the past years, the variation of displacements measured is still to be significantly larger, and sometimes much smaller, than those calculated. This is because of the inappropriate selection of stiffness parameters in characterization of soil settlement [1]. The linear elastic theory was the base of solutions in characterization of settlement (ε) of coarse-grained soils, which depends on the elastic modulus of a soil (E) and the flexibility of the resting foundation as:

$$\varepsilon = \frac{\Delta\sigma}{E}. \quad (1)$$

Most soils behave non-linearly under compression upon subjected to a stress change. As such, the linear elastic theory cannot be suitable, especially when dealing with compressible soils such as fine-grained soils (more specifically clays). In this case, the settlement can be found by:

$$\varepsilon = \frac{C_c}{1+e_o}\log\frac{\sigma'_f}{\sigma'_o}, \quad (2)$$

where σ'_o and σ'_f represent the initial and final effective stress, respectively.

The equation (2) shows that the settlement of a fine soil due to primary consolidation is a function of two key parameters; the compression index C_c, and the initial void ratio e_o, which can be determined from the traditional one-dimensional compression laboratory test. A range of theoretical, empirical, and semi-empirical relationships for C_c have been developed (e.g. [2]–[8]) depending on one (e.g. liquid limit, water content, field void ratio) or more soil properties. However, most civil engineers and soil researchers often report the values of C_c only, ignoring the magnitude of e_o. Sometimes, the reported e_o is determined from a soil sample different to that used in determining the C_c, which is not acceptable. The challenge of using two compression parameters is avoided by using soil modulus, the constrained modulus, the slope of stress-strain relationship which was first introduced by N. Janbu [9], using two non-dimensional parameters; a stress exponent, d, and a modulus number, m, as follows:

$$\frac{d\sigma}{d\varepsilon} = m\sigma_o\left(\frac{\sigma'}{\sigma_r}\right)^{1-d} = M, \quad (3)$$

where M is, σ' is an effective stress (kPa); and σ_r is a reference stress equal to (100 kPa).

Soil moduli have been used by several researchers as a measure of soil stiffness (e.g. [10]–[16]). Measurements of soil modulus by means of laboratory tests are quite difficult and often inaccurate because of the significant disturbance [17]. Therefore, correlations were always required to estimate the specific soil modulus. The constrained modulus approach for settlement, equation 3, is used where is no strain in the perpendicular direction which is the case of the soil behavior under one-dimensional compression. It combines the principles of linear and non-linear behavior [9]. It is also applicable for all types of soils even intermediate soils (e.g. silty sand, clayey sand) that exhibit transitional behavior which are not addressed properly by the traditional approaches (equation 1 and 2) [9] and [18]. According to Byington [19], the traditional approaches are limited and cannot be used with silt containing 15 % or less clay content. The interaction and arrangement of the fine and coarse particles in these soils affects the compressibility behavior such that they can be expected to exhibit behaviors and modes associated with clay and sands soils.

Despite the applicability of the constrained modulus approach, the approach is not commonly used in soil mechanics for soil compressibility and settlement. Very few studies were undertaken and provided preliminary information. N. Janbu [9] was the first who used the modulus approach to describe the compressibility behavior of rock, sand, silt and clay soils tested by oedometer and triaxial. He showed that the approach can describe the compressibility behavior provided that the accurate values of soil modulus number and stress exponent are selected. Schanz and Vermeer [1] tested the stiffness of three types of sands – Toyoura, Karlsruhe and Hostun sands – using the oedometer and triaxial modulus and showed that both normalized moduli have nearly same values.

The modulus number and stress exponent are of high importance in modulus-based settlement calculation. According to N. Janbu [9], the stress component (d) and the modulus number (m) both varies with soil initial porosity (n), the coarser the soil, the larger the stress exponent and modulus numbers assumed that, d = 1 for non-plastic soils, and d = 0 for plastic soils. The variation range of the modulus number, m, is really quite wide, from 6 orders of magnitude between coarse- and fine-grained soils.

According to the Canadian Foundation Engineering Manual 1992, the value of m for coarse soils, gravels and sands, ranges between 100 and 400 and for fine soils, silts, and clays, it is between 5 and 200. These values of m are approximate, and for intermediate soils such as silty clays or sandy clays, the values will be different as the compressibility behavior is different to that in pure soils. Therefore; the objective of this study is to determine both the modulus number, m , and the stress component, d , of 133 fine and mixed soils using the constrained modulus-based approach, and also to establish a new empirical relationship for the modulus number, m .

1.1 Soil settlement determination

With the knowledge of soil modulus number, m , stress exponent, d , and effective stress, σ' , soil settlement can be determined. If, by the assumptions of N. Janbu [9], the stress exponent is fixed to unity for very coarse soils, rocks and gravels, and integrating the equation (3) gives:

$$\varepsilon = \frac{\sigma'_f - \sigma'_o}{100m}. \tag{4}$$

For fine soils, it equals to zero and equation (3) be:

$$\varepsilon = \frac{1}{m} ln\left(\frac{\sigma'_f}{\sigma'_o}\right). \tag{5}$$

However, for intermediate, composite, and transitional soils such as sandy clays, sandy silts and silty sands that are not covered properly by the traditional approaches, an intermitted value corresponds to 0.5 can be used which lies in an agreement with the fact that for each stress increment, the strain of a soil gets gradually smaller

$$\varepsilon = \frac{1}{5m}\left(\sqrt{\sigma'_f} - \sqrt{\sigma'_o}\right). \tag{6}$$

To estimate the modulus number, m , the normal compression curves $\left(e - log\,\sigma'\right)$ of soils were first converted to constrained modulus $\left(M - \sigma'\right)$ using the definition of volume compressibility $\left(m_v\right)$ as:

$$M = \frac{1}{m_v} = \frac{\delta\sigma'}{\delta e}\frac{(1+e)}{1000}. \tag{7}$$

Then, applying the best fit to the equation (3).

2. Materials and Methods

It is known that soil type, stress history and magnitude of effective stress play an important role in compressibility behavior of soils so a range of data from different sources varying in geological origin, soil type, sample preparation and index properties was selected in analysis. Published data of [5–7, 20–31] was included in the analysis. The soil samples in these sources are natural, remolded, and reconstituted samples prepared with an initial water content of 1–2 times of liquid limit. The standard one-dimensional oedometer test was used in these papers to assess the compression characteristics with a stress varying between (2.5–40000 kPa). Table 1 summarizes the soil data used in the analysis.

Table 1. Data used in analysis.

No.	Soil type	IL	m	d	Sources
1	Sail soil	158.3	5	0	[5]
2	Black cotton soil	97.3	5.14	0	
3	Red soil	45.3	10.16	0	
4	Vienna clay	47	9.8	0	
5	Silty clay	56.7	11.9	0.01	
6	Silty sand	36.2	16.4	0.01	
7	10 % bentonite + 90 % brown soil	68	8.5	0.13	[20]
8	20 % bentonite + 80 % brown soil	77	8.2	0.25	
9	10 % bentonite + 90 % black cotton	88	5.8	0.18	
10	20 % bentonite + 80 % black cotton	93	4.97	0.15	
11	30 % bentonite + 70 % brown soil	95	4.58	0.15	
12	30 % bentonite + 70 % black cotton	104	4.79	0.17	
13	49 % bentonite + 51 % sand	162	3.6	0.19	
14	59 % bentonite + 41 % sand	195	3.77	0.24	
15	100 % bentonite	330	3	0.28	
16	Lower Cromer Till	25	14.5	0.15	[21]
17	Boulder clay	28	18.8	0.13	
18	Silty clay	28	17.5	0.10	
19	Magnus Clay	35	9.68	0.11	
20	Grangemouth	35	10.7	0.1	
21	Ton V	36	10.33	0.1	
22	Weald clay	39	10.95	0.08	
23	Boston blue clay	39	12.7	0.07	
24	Red soil	45.3	9.8	0.06	
25	River Severn alluvium	46	12.7	0.05	
26	Wiener Tegel	46.7	9.3	0.04	
27	Oxford clay	53	9.7	0.03	
28	Ton IV	58	9.14	0.02	
29	Residual clay	58	8.91	0.01	
30	London Clay	62.3	7.3	0.02	
31	London Clay	67.5	6.68	0.03	
32	Ganges delta clay	69	7.8	0	
33	Gosport clay	76	6.8	0.02	
34	London Clay	77	6.9	0.02	
35	Brown London Clay	88	6.1	0.03	
36	Black cotton clay	97.3	5.8	0.04	
37	Kleinbelt Ton	127	5.1	0.06	
38	Argile plastique	128	6.3	0.03	
39	SAIL	159.3	5.3	0.05	
40	Red earth 1	37	12.867	0.03	[6]
41	Silty soil	39	13.797	0.12	
42	Kaolinite 1	48	9.88	0	
43	Red earth 2	48	9.88	0	
44	Kaolinite 2	55	12.449	0.13	
45	Cochin clay	56.4	10.57	0.18	
46	Brown soil 1	58.5	8.23	0.1	
47	Illatic soil	73.4	10.81	0.13	
48	BC soil	73.5	5.39	0	
49	Glacial sity clay	28	21.625	0.16	
50	Boulder clay	28	19.493	0	

No.	Soil type	IL	m	d	Sources
51	Sandy delta mud	36	8.195	0	
52	Weiner tegel	46.7	9.775	0.02	
53	Vienna clay	47	10.5	0.04	
54	Oxford clay	53	10.465	0	
55	Residual clay	58	9.82	0.08	
56	London clay	77	7.3	0.03	
57	Kleinbelt Ton	127	5.57	0.01	
58	MX80 clay	520	1	0	[22]
59	Kunigel clay	474	1.2	0	
60	Fourges clay	112	3.9	0.04	
61	96 sand + 4 clay	33	20.4	0.24	[23]
62	92 sand + 8 clay	37.5	16.7	0	
63	75 sand + 25 clay	47	10.39	0	
64	55 sand + 45 clay	61	9.8	0.16	
65	Na-Ca MX80	520	1	0	[24]
66	Na-Kunigel	474	1.2	0	
67	Atchafalaya	101	6.2	0.1	[25]
68	Attapulgite	202	5	0	
69	Boston blue clay	45	13.9	0.05	
70	Kaolinite	42	15.8	0	
71	Kaolinite (Ca 0.01 M)	81	7.4	0.2	[26]
72	Bentonite (Ca 0.001 M)	102	3.8	0.1	
73	SPV200 WB	354	1.7	0	[27]
74	Dunkettle silt 1	36	18	0.14	[28]
75	Dunkettle silt 2	40	16	0.36	
76	Sligo sandy silt	61	12	0.24	
77	Dunkettle silt 3	22	24	0.3	
78	Dunkettle silt 4	20	27	0.5	
79	0 sand + 100 clay (bentonite 9 : kaolin 1)	260	3.6	0.1	[29]
80	0 sand + 100 clay (bentonite 7 : kaolin 3)	198.4	3.9	0.13	
81	30 sand + 70 clay (bentonite 9 : kaolin 1)	155	4.4	0.28	
82	0 sand + 100 clay (bentonite 5 : kaolin 5)	157.2	4	0.1	
83	40 sand + 60 clay (bentonite 9 : kaolin 1)	127.7	4.8	0.2	
84	30 sand + 70 clay (bentonite 7 : kaolin 3)	123.7	4.9	0.26	
85	40 sand + 60 clay (bentonite 7 : kaolin 3)	106.3	4.78	0.26	
86	30 sand + 70 clay (bentonite 5 : kaolin 5)	101.2	5.15	0.2	
87	50 sand + 50 clay (bentonite 5 : kaolin 5)	68.8	5.94	0.17	
88	Kaolinite	29.1	13.2	0.13	[7]
89	95 % kaolinite + 5 % bentonite	43.9	9.2	0.11	
90	90 % kaolinite + 10 % bentonite	53.3	7.8	0.06	
91	85 % kaolinite + 15 % bentonite	61.7	4.9	0	
92	Lianyungang clay	74	5.5	0	[30]
93	Baimahu clay	91	4.6	0	
94	Kemen clay	61	7.6	0.04	
95	Kaolinite	56	9.55	0.44	[31]
96	10 fine sand – 90 kaolinite	52.1	10.91	0.48	
97	20 fine sand – 80 kaolinite	48.2	11.40	0.50	
98	30 fine sand – 70 kaolinite	43.4	11.69	0.50	
99	40 fine sand – 60 kaolinite	38.7	12.91	0.51	
100	50 fine sand – 50 kaolinite	33.2	14.80	0.57	
101	60 fine sand – 40 kaolinite	27.9	15.98	0.62	

No.	Soil type	IL	m	d	Sources
102	Kaolinite	56	9.55	0.10	
103	10 medium sand – 90 kaolinite	51.6	10.24	0.48	
104	80 medium sand – 20 kaolinite	47.3	10.02	0.53	
105	70 medium sand – 30 kaolinite	42.6	10.37	0.57	
106	60 medium sand – 40 kaolinite	37.8	12.59	0.52	
107	50 medium sand – 50 kaolinite	32.2	13.39	0.58	
108	60 medium sand -40 kaolinite	26.9	15.18	0.60	
109	10 fine sand – 90 bentonite	97.1	5.06	0.03	
110	20 fine sand – 80 bentonite	87.9	4.99	0.12	
111	30 fine sand – 70 bentonite	78.1	5.01	0.22	
112	40 fine sand – 60 bentonite	67.5	5.50	0.33	
113	50 fine sand – 50 bentonite	58.2	6.16	0.42	
114	60 fine sand – 40 bentonite	47.3	7.65	0.53	
115	10 medium sand – 90 bentonite	95	4.90	0.05	
116	20 medium sand – 80 bentonite	85.8	4.99	0.14	
117	30 medium sand – 70 bentonite	76.5	4.97	0.24	
118	40 medium sand – 60 bentonite	65.7	5.75	0.34	
119	50 medium sand – 50 bentonite	56	7.08	0.44	
120	60 medium sand – 40 bentonite	45.3	11.22	0.55	
121	10 medium sand – 90 sepiolite	104.9	5.50	0.00	
122	20 medium sand – 80 sepiolite	92.9	6.31	0.07	
123	30 medium sand – 70 sepiolite	81.5	6.61	0.19	
124	40 medium sand – 60 sepiolite	70	7.59	0.30	
125	50 medium sand – 50 sepiolite	58.5	7.94	0.42	
126	60 medium sand – 40 sepiolite	48	8.43	0.52	
127	70 medium sand – 30 sepiolite	36.5	12.59	0.56	
128	Illlite	37.6	12.70	0.57	
129	20 fine sand – 80 Illite	31.5	12.38	0.59	
130	40 fine sand – 60 Illite	26	13.22	0.6	
131	Bentonite 1	112	3.7	0.04	Present study
132	Sepiolite 1	125	5	0.14	
133	Attapulgite 1	217	4.5	0.12	

In addition, three common commercially clays; bentonite, sepiolite, and attapulgite were chosen in the present study which covers a range of plasticity (105 % ≤ IL ≤ 220 %). The index properties, determined by ASTM D4318–17e1 [32], D854–14 [33], and D2487–17 [34] of the materials are shown in Table 2. The plasticity chart of the data used is shown in Fig. 1. As per Fig. 1, the classification of clays is high plasticity silts (MH). Also, the used data covers a range of clayey, silty, and intermediate soils with high and low plasticity which is very important as to cover a range of soils for settlement analysis.

The clays were prepared in a blender as slurry by adding de-aired water using 1.75 times of liquid limit ensuring a fully saturated and homogeneous sample. The mixture was poured into a rigid stainless steel consolidometer tube with a 14 cm diameter and 25 cm height and then consolidated to the desired stress. The compressibility test was conducted using the standard oedometer equipment following ASTM D2435/D2435M-11 [35] procedures. The internal area of the consolidation ring was lubricated with silicon grease so that the side friction can be minimized, and the test was carried out at maintained room temperature of 20 °C. The ring was inserted into the consolidated sample to bring the target sample. The assembled cell was mounted and placed on the platform of the oedometer equipment with an adjusted dial gage of a 0.001 mm resolution. Before applying loads, the soil sample was kept in saturated condition by submerging it in de-aired water for the whole test period. Increments of load increments were applied up to maximum stress of 1280 kPa and each increment was doubled when the primary compressibility for the current load was completed.

Table 2. The properties of materials used.

Clays	Atterberg limits	Specific gravity (G_s)	Classification
Bentonite 1	IL = 112 % PI = 52 %	2.64	MH
Sepiolite 1	IL = 125 % PI = 29 %	2.38	MH
Attapulgite 1	IL = 217 % PI = 101 %	2.12	MH

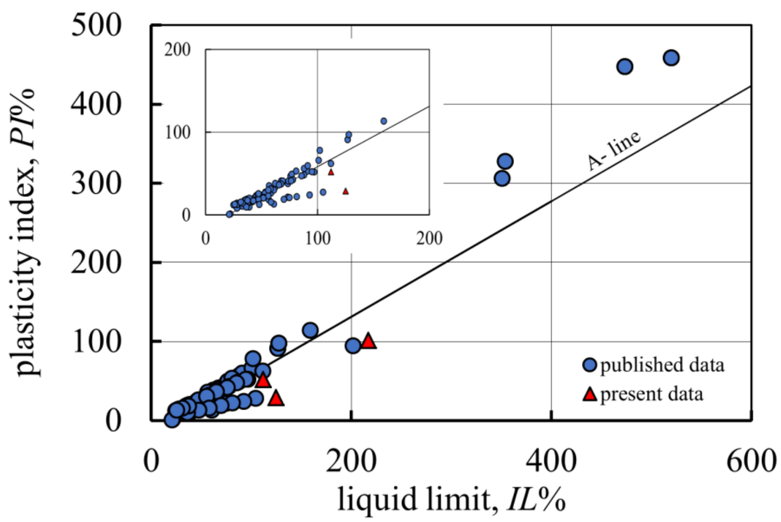


Figure 1. The plasticity chart of present clays and data used.

3. Results and Discussion

Fig. 2 shows the normal compression curves of the bentonite, sepiolite and attapulgite clays that were tested in the present study. It can be seen that the curves are slightly concave up for the stresses larger than 10 kPa which is the most common.

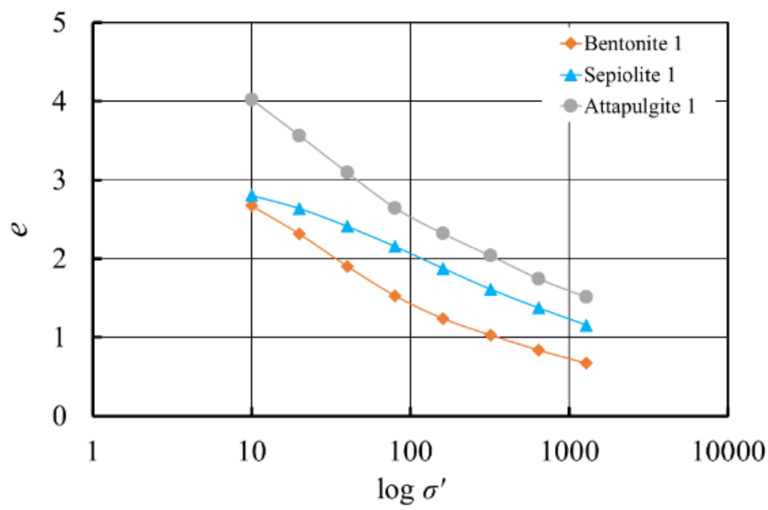


Figure 2. Normal compression curves of clays (bentonite, sepiolite, and attapulgite).

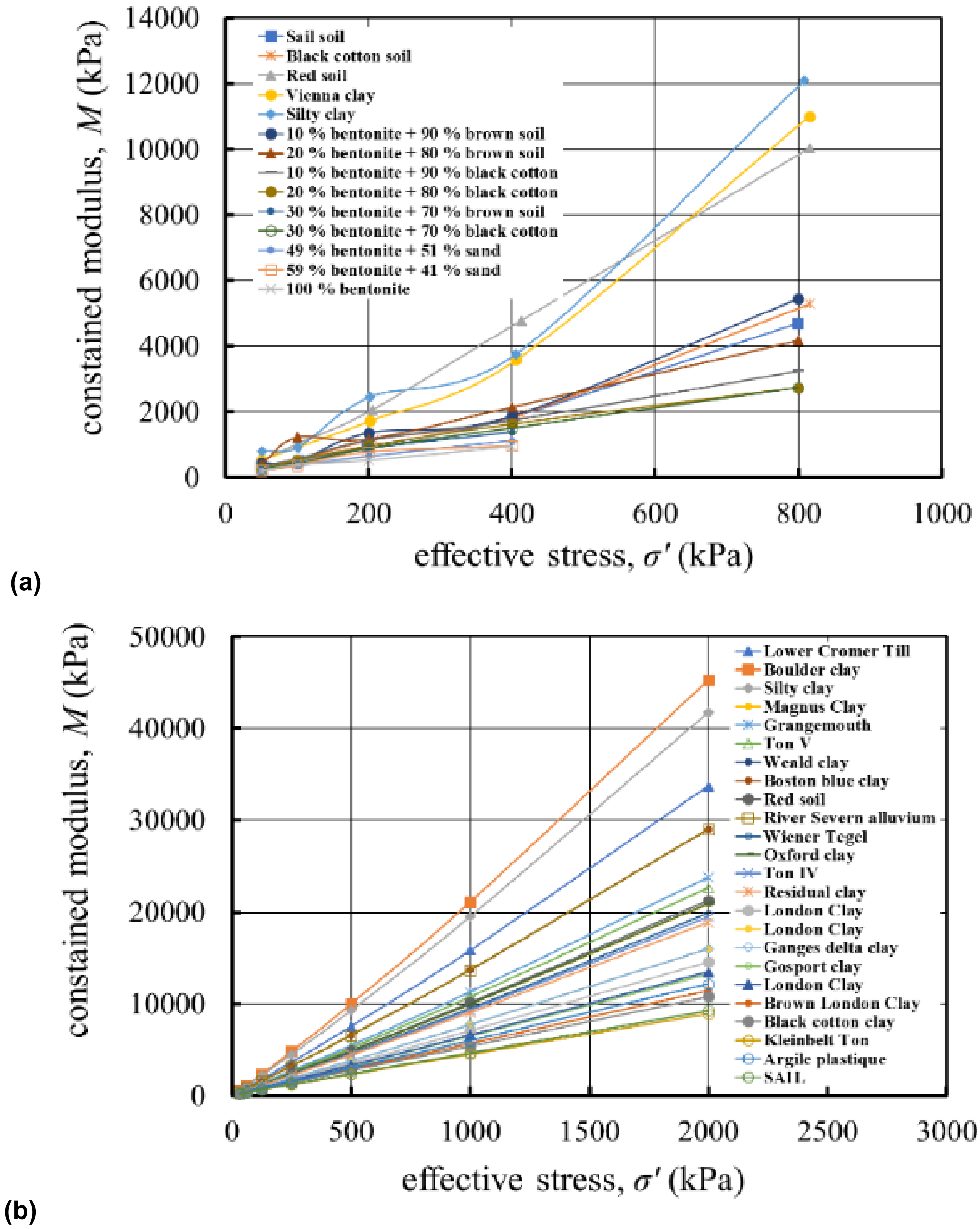
The normal compression curves in the preset study (Fig. 2) and in previous selected studies were replotted in terms of the constrained modulus, M, versus effective stress, σ' (as seen in Fig. 3). It can be seen that the constrained modulus is a function of effective stress, stress history, and soil type. The modulus increases when the stress increases.

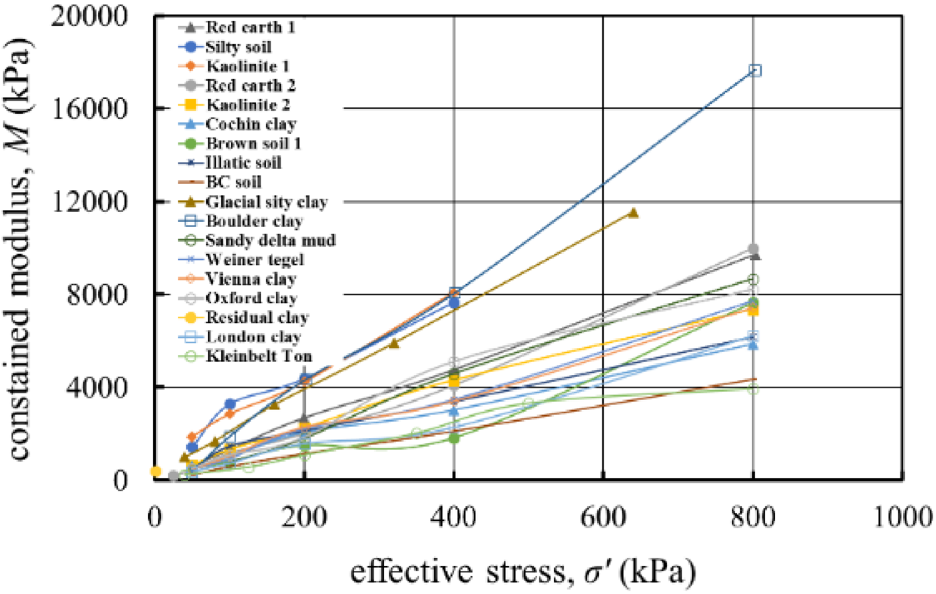
The modulus number, m, and stress exponent, d, were determined using the best-fit of the equation (3). The values are presented in Table 2 and correlated in Fig. 4 and 5 with liquid limit, IL. It can be observed that there is a good relationship for m with IL with $R^2 = 0.83$ in the form:

$$m = 264.11(IL)^{-0.841}. \quad (8)$$

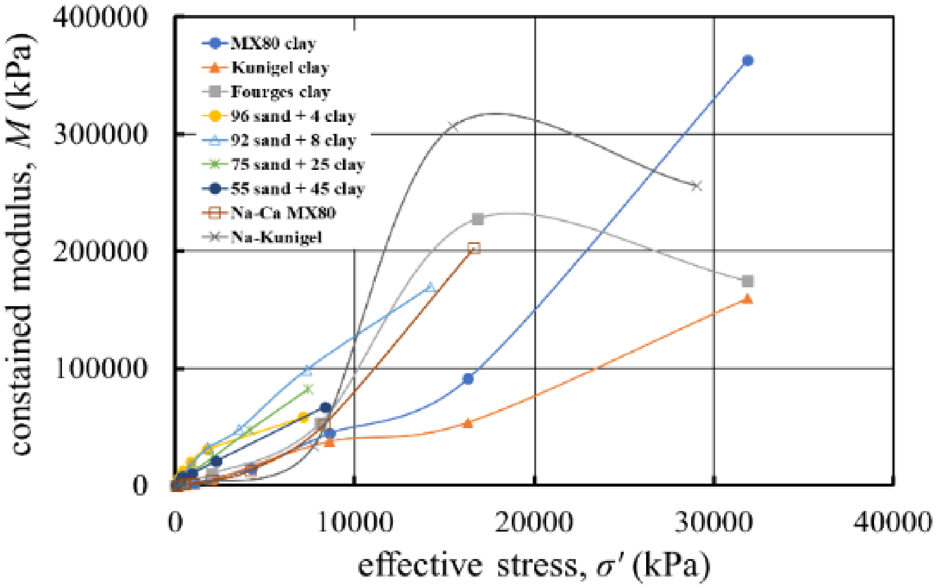
This is acceptable because the liquid limit is a material property that is relied on soil composition, particle size and surface characteristics. This single relationship has an advantage of the fact that liquid limit test is easy to conduct in lab.

The stress exponent, d, shows a scatter as it depends on soil plasticity and particle size distribution and varies from 0 for clayey soils to 0.3–0.6 for silty and clayey sand soils. This is similar to the assumptions of N. Janbu [9] for normal consolidated clays and intermediate soils.

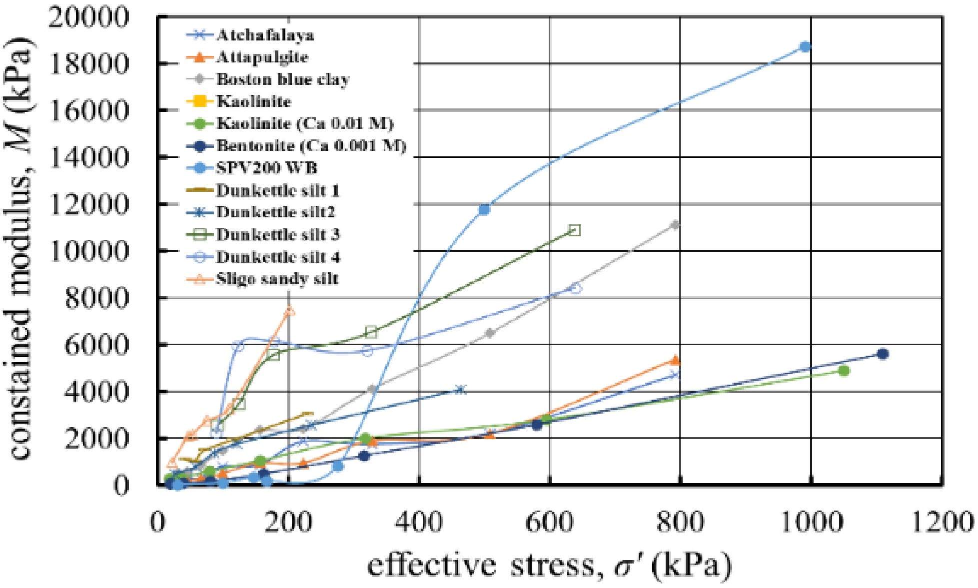



constained modulus, M (kPa)
effective stress, σ′ (kPa)
20000
16000
12000
8000
4000
0
0
200
400
600
800
1000
Red earth 1
Silty soil
Kaolinite 1
Red earth 2
Kaolinite 2
Cochin clay
Brown soil 1
Illatic soil
BC soil
Glacial sity clay
Boulder clay
Sandy delta mud
Weiner tegel
Vienna clay
Oxford clay
Residual clay
London clay
Kleinbelt Ton

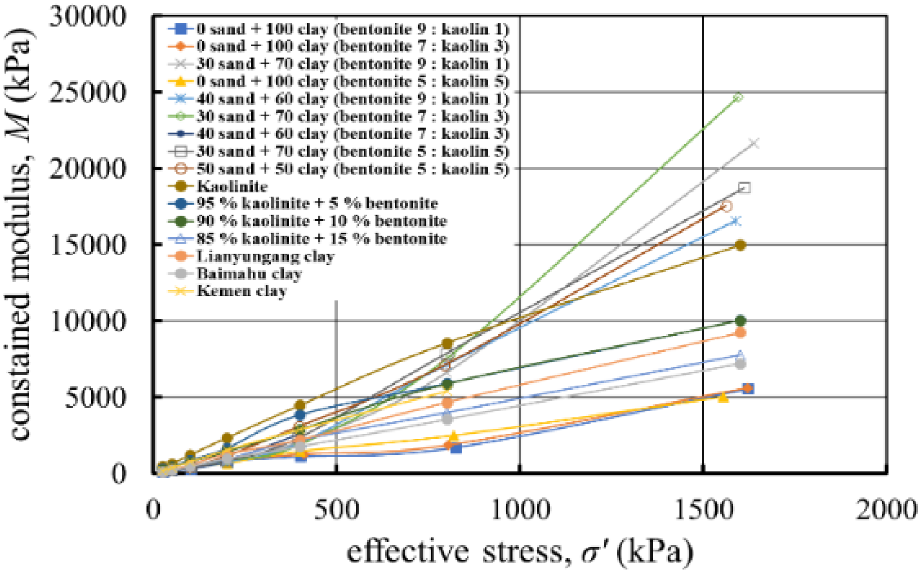
(c)


constained modulus, M (kPa)
effective stress, σ′ (kPa)
400000
300000
200000
100000
0
0
10000
20000
30000
40000
MX80 clay
Kunigel clay
Fourges clay
96 sand + 4 clay
92 sand + 8 clay
75 sand + 25 clay
55 sand + 45 clay
Na-Ca MX80
Na-Kunigel

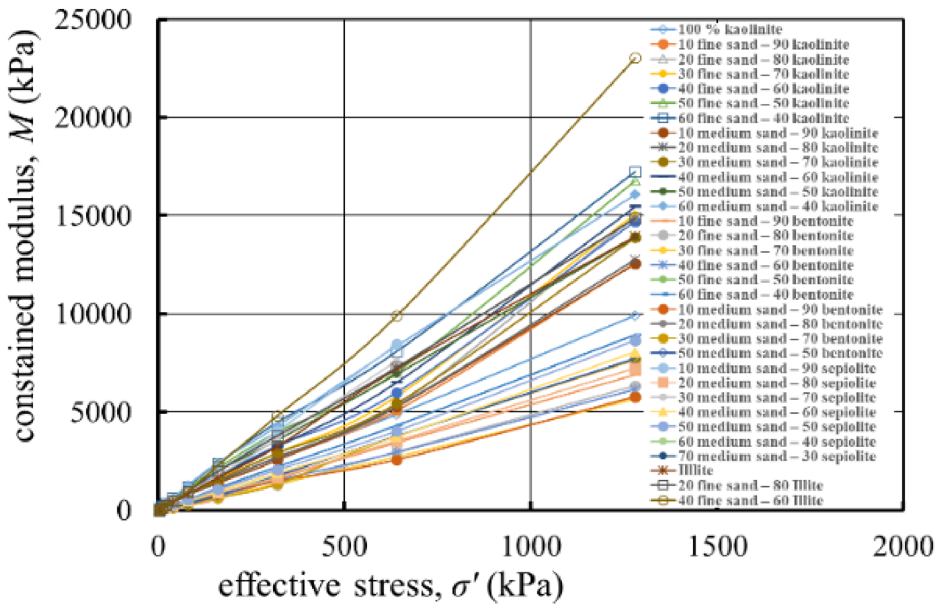
(d)


constained modulus, M (kPa)
effective stress, σ′ (kPa)
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
0
200
400
600
800
1000
1200
Atchafalaya
Attapulgite
Boston blue clay
Kaolinite
Kaolinite (Ca 0.01 M)
Bentonite (Ca 0.001 M)
SPV200 WB
Dunkettle silt 1
Dunkettle silt2
Dunkettle silt 3
Dunkettle silt 4
Sligo sandy silt

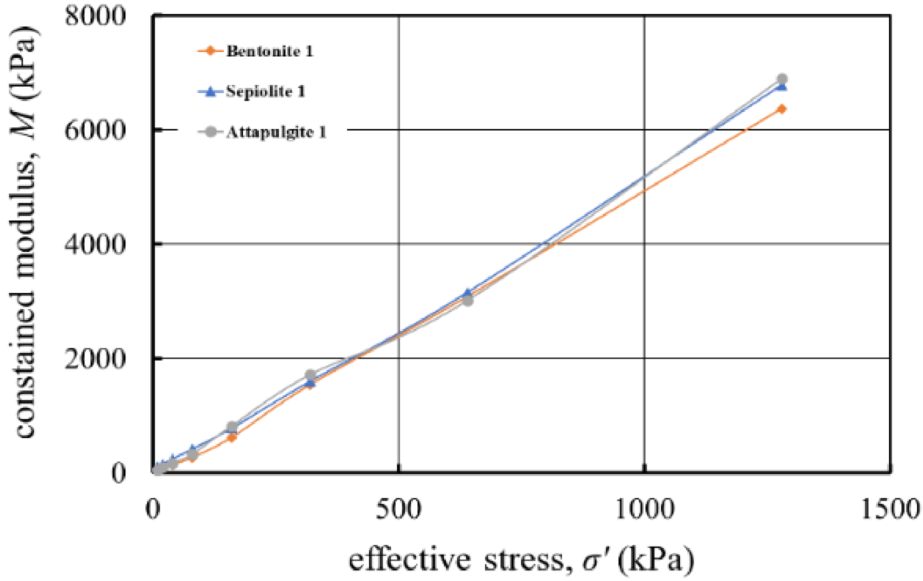
(e)



(f)



(g)



(h)

Figure 3. The relationship between the constrained modulus, M, and effective stress for soils: (a) [5] & [20]; (b) [21]; (c) [6]; (d) [22–24]; (e) [25–28]; (f) [29–30]; (g) [31]; (h) present clays.

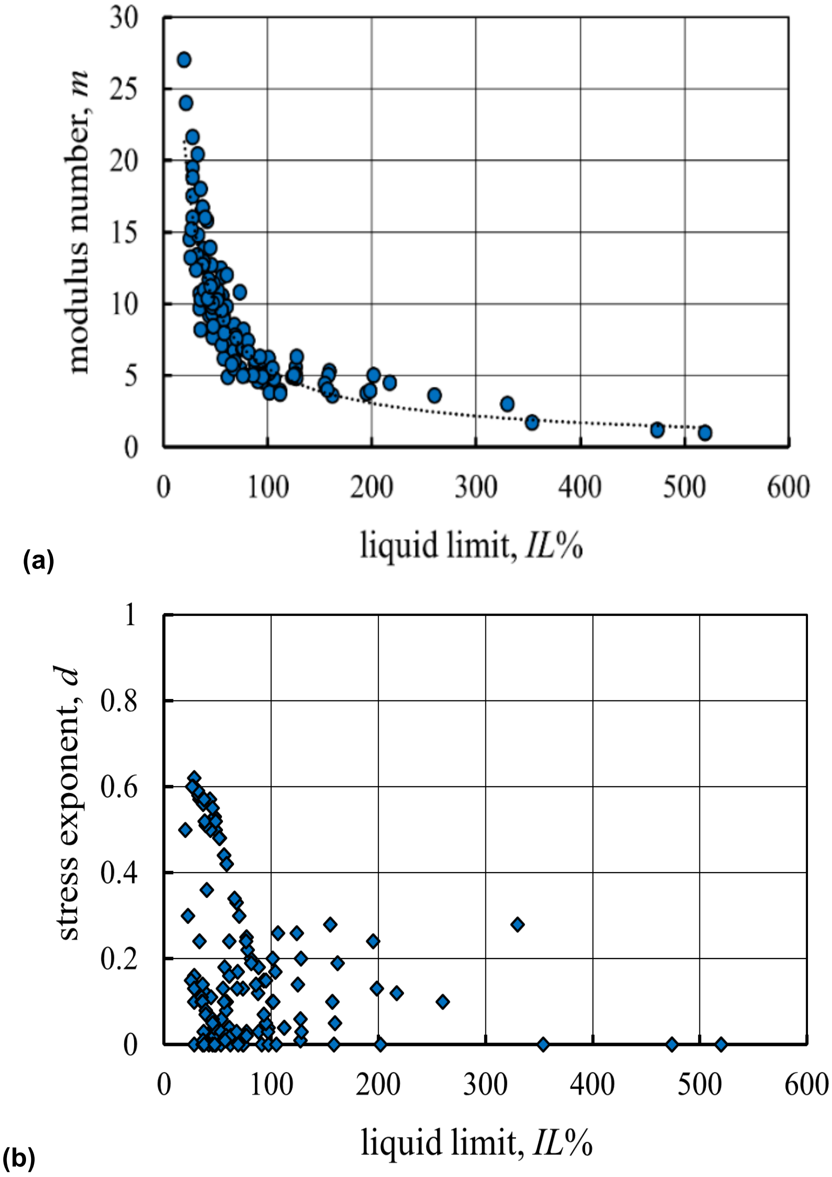


Figure 4. The relation between: (a) modulus number, m; (b) stress exponent, d, with liquid limit, IL.

4. Conclusion

This study is devoted to exploring the suitability of the constrained modulus approach for soil settlement determination. An analysis of a range of compressibility data taken from different high-quality papers along with the present experimental results on three commercial clays (bentonite, sepiolite and attapulgite) using the modulus-based approach of N. Janbu shows that:

1. The constrained modulus-based approach is a convenient approach to adequately describe the settlement phenomena of fine and intermediate transitional soils using the stress exponent (d) and the modulus number (m).
2. The results show that the stress exponent (d) in the approach depends on soil plasticity and particle size distribution and varies from 0 for clayey soils to 0.3–0.6 for silty and clayey sand soils.
3. There is a relationship between the soil modulus number (m) and the liquid limit (IL) in that an increase in the liquid limit leads to a decrease in the modulus number.
4. An empirical relationship for modulus number, m, based on the soil liquid limit, IL, was established with $R^2 = 0.83$.

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