Application of "Big Data" to Engineering Properties of Hong Kong Soils

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Abstract

In recent years, big data is becoming a very powerful tool in processing extremely large amount of data in such fields as finance, industry, engineering, etc. For geotechnical engineering, large number of laboratory and in-situ tests (mostly SPT) have been carried out in the past few decades. Laboratory testing includes soil classification and most importantly three major engineering properties: shear strength, compressibility and conductivity. In order that these data forming the big data can be useful in engineering design, a lot of processing/analysing works have been carried out and these indicate that soil type is the most dominant parameter affecting all engineering properties. Within each soil type, there are some secondary factors such as fines content, dry density, etc, which have only a secondary effect on these properties. Another dominant primary factor is SPT, which will affect most importantly the shear strength. A 2020 HKIE paper by the author has established that the relationship between shear strength and SPT is unique, irrespective of the soil type. As for compressibility, SPT is in general directly proportional to the elastic modulus. However, SPT has basically no effect on conductivity. Finally, the method of entering the processed data is proposed.

Keywords: Big data, Soil properties, Hong Kong

1 Introduction

Pioneering work on soil testing was carried out in the 1950s to 1960s by the late Professor Lumb in the soil laboratory of University of Hong Kong. In spite of the limited testing facilities and number of tests carried out in that period, these works do lay out a solid foundation for future testing works in Hong Kong.

Since the late 1970s, a lot of site investigation and laboratory testing have been carried out in various Hong Kong testing laboratories, following in general the guidelines set out by Professor Lumb. By that time, all the testing procedures and testing facilities have been basically standardized. Accordingly, data thus collected should be useful for big data analysis.

For big data analysis to be applicable to engineering properties of Hong Kong soils, the following problems have to be considered: firstly the types of soils to be considered in Hong Kong; secondly the major types of engineering properties to be considered for these soils; thirdly, the major primary and secondary parameters affecting these properties and their respective correlations and finally the type of processed data to be entered and methods of determining design parameters from such data.

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2 Hong Kong Soils: their Formation and Major Types

2.1 General

To be exact from a geological point of view, the earth's crust consists of various types of rock materials. Soil materials are just some special types of rock materials formed as a result of weathering or erosion of the parent rocks or other soils. Accordingly, there are only two general soil types: weathered soils and transported soils.

2.2 Weathered Soils

These are formed as a result of weathering of the parent rocks in-situ. The two major rock types in Hong Kong are 'Granite' and 'Volcanic' which were formed from the large scale volcanic activities during the Middle Jurassic period to Early Cretaceous period (these two periods being formed respectively about 210 and 140 million years ago), thus forming the volcanic rock. This is followed by intrusion of the volcanic magma, thus forming the granitic rock. It is obvious from the above processes that these two have no significant difference in mineral content. The only difference is grain size, with granite minerals being coarse sand size and volcanic minerals in the ground-mass of medium to fine silt size as a result of faster rate of cooling in the outer volcanic rock.

2.3 Transported Soils

These are formed as a result of the erosion of weathered soils (or rocks) or other transported soils by various erosion agents. During this erosion process, the soil particles are detached from each other, transported and finally re-deposited again to a soil layer with compactness and particle size depending on the extent of the above processes of detachment, transportation and re-deposition. In some cases, further weathering or chemical and physical changes might also take place after re-deposition. The extent of these processes of detachment, transportation and re-deposition is, in turn dependent entirely on the type of erosion agents, thus forming different types of transported soils.

2.4 General Classification

In view of the above considerations, Hong Kong soils can be further classified into the following 6 major types:

Weathered soils:

(1) Completely/highly decomposed granite (C/HDG)

(2) Completely/highly decomposed volcanic (C/HDV)

Transported soils:

(3) Fill/colluvium in granitic areas

(4) Fill/colluvium in volcanic areas

(5) Marine deposit (MD)

(6) Alluvial deposit (ALL)

2.5 Classification in Accordance to Formation Process

Finally, purely from the formation process, these soils should be classified into three distinct groups: (a) weathered soil comprising mostly C/HDG and C/HDV; (b) fill/colluvium in both granitic and volcanic areas; (c) MD and ALL. Each of these three groups has a different formation process, with the first one formed from in-situ weathering of the parent rocks and the other two being transported soils formed from erosion of the weathered soils (or rocks) or other transported soils.

The erosion process for fill/colluvium in both areas is vastly different from that of MD and ALL. Firstly, MD and ALL have a longer period of dislocation, transportation, sorting and re-deposition, in particular the re-deposition period. In view of the above phenomena, MD and ALL have more intensive chemical and physical changes and weathering, the only exception being some of the more recently deposited ALL.

3 Engineering Properties

3.1 General Considerations

In general, there are three major engineering properties: shear strength, compressibility and conductivity. Shear strength, in classical soil mechanics, concerns the determination of shear strength parameters: angle of internal friction (ϕ) and cohesion (c). Both parameters can be expressed in term of total or effective stresses. Compressibility in its simplest form can be represented by the SPT value in-situ, or by the consolidation testing results in the laboratory. Conductivity is represented most directly by its permeability value (k). Another parameter is the coefficient of consolidation (c_v) introduced by Terzaghi but this also measures the soil compressibility.

Of the three major engineering properties, the dominant parameter affecting them is the soil type: weathered or transported soils. In view of the very different formation processes between weathered and transported soils as well as between various transported soils, the engineering properties of different soil types can vary very substantially, in particular the conductivity property, with c_v and k varying up to ten thousand times or more, as indicated in the paper by Wong (2020a).

Another dominant primary factor is the SPT value, which will affect most importantly the shear strength. In a recent paper by Wong (2020b), it has been established that for Hong Kong soils, there is a unique relationship between soil shear strength and SPT, irrespective of the soil type. Quite unlike conductivity, the variation in term of shear strength is at most a couple of times among different soil types.

As for compressibility, the more compressible the soil, the lower is the SPT. With SPT varying from less than 1.0 for MD to over 100 for C/HDG (see Tables 2 & 3 in Wong (2020b)), the variation in the worst case scenario might be up to several hundred times. However, the above trends do not hold for conductivity. Strictly speaking, there is in general no meaningful correlation between SPT and conductivity.

Within each soil type, there are some secondary factors such as fines content (F), dry density (τ_d), etc, which can also affect the soil engineering properties. However, these effects are of a more secondary nature as evidenced by the experimental fact that two soil types with similar fines content and dry density can have quite different engineering properties. The same applies to the other secondary factors.

So far research on soil engineering properties does indicate that these are dominated by the two primary factors: soil type and SPT. Accordingly these must be firstly identified in future processing work forming the big data. These together with other secondary factors can fine-tune the engineering properties for design. For large/important jobs, some confirmatory testing might be necessary.

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3.2 Soil Shear Strength

3.2.1 Determination By Laboratory Testing

It should be noted that the earlier developed laboratory testing methods are for the undrained conditions. This is not surprising as at that time the effective stress principle has not yet been proposed. It is only in the triaxial compression testing (e.g., Bishop and Hankel, 1962) that both undrained and drained conditions can be adopted. As for in-situ testing, most of them are for the undrained conditions. Table 1 summarizes the various laboratory and in-situ testing methods for determining soil shear strength.

Table 1. Laboratory and in-situ testing methods for determining soil shear strength parameters

	Testing method commonly adopted		Drainage condition	Soil shear strength parameter determined
Laboratory	Triaxial consolidation	(1) CQP_M CQP_S CD_M CD_S	Drained and undrained	ϕ' angle of internal friction c' cohesion
	Triaxial compression	(2) UU UC	Undrained	ϕ_u undrained friction angle c_u undrained cohesion
	Vane shear test		Undrained	c_u undrained cohesion
	Shear box	Robertson, Golder, Simple shear box	Undrained	ϕ friction angle along joint surface separating 2 soil/rock blocks
		Large shear box	(3) Drained and undrained	ϕ friction angle of rockfill
In-situ	SPT (Standard Penetration Test)		Undrained	Correlating with dry density and hence ϕ' & c'
	GCO probe test			Similar to SPT but limited to fill/colluvium for only a few metres.
	Vane shear test		Normally testing under undrained condition unless testing rate reducing to required low value	c_u undrained cohesion
	CPT (Cone Penetration test)			Theoretically ϕ' and c' can be determined, but accurate measurement & interpretation being difficult in practice.
	Plate loading test			
	Pressuremeter test			
	In-situ dry density		(4) (not applicable)	Correlating with ϕ' and c'

Notes: 1. CQP = Consolidated undrained with pore pressure measurement CD = Consolidated drained
M = Multi-stage S = Single stage
2. UU = Unconsolidated undrained UC = Unconfined compression
3. Drainage condition not as well controlled as in triaxial testing.
4. Operation involving mainly sampling and always above ground water table.

A very vital point to note in choosing the most appropriate testing method of analysis and interpretation is the type of soil: granular or cohesive. When first applying the triaxial testing by Bishop and Hankel (1962), this is mostly for saturated cohesive clay with only a single stage testing. According to Table 1, the type of testing will be CQP_S and CD_S for measuring ϕ' and c'. As for c_u this will be measured by UU or UC. For weathered soils and fill/colluvium in both areas, UU or UC testing is basically not applicable.

In view of the fairly large sample variation for Hong Kong soils, Lumb (1964) introduced the multi-stage testing and this was later further modified by Wong (1978). Alternatively, single stage testing (e.g. Beattie and Chau, 1978) has also been proposed and in this case the "Method of Least Squares" has been adopted to determine the most appropriate shear strength parameters. Wong (1982) summarised the various then existing triaxial consolidated testing methods, comprising: CQP_M, CQP_S, CD_M and CD_S as in Table 1. He concluded that CQP_M being the most feasible one with respect to both the quality of the testing results as well as for testing cost.

Regarding method of analysis, Wong (1982) recommended that for basically granular materials soil shear strength parameters should be determined from the arithmetic mean of all individual CQP_M test results, each with modification to account for the decreasing cohesion with each stage of shearing as suggested by Wong (1978). As for basically cohesive materials, the undrained cohesion is directly measured from the test results and no further analysis is required.

Since the late 1970s to early 1980s, most of the testing and analysing works are based more or less on the above papers. In addition, there is also a trend of plotting all the triaxial test results on a combined deviator stress (q) versus effective mean normal stress (p') plot to assess the most likely soil shear strength parameters (ϕ', c') visually, in which ϕ' is related to c' by the Mohr-Coulomb criterion.

It is not until 2020 then a new method other than those previously discussed has been introduced by Wong (2020b) to analyse the numerous triaxial testing results so far collected and processed in the past few decades. In contrast to most previous methods of analysis, it is the median (ϕ', c') values instead of the mean (ϕ', c') values that will be adopted. By definition, any design line specified by the set of (ϕ', c') values will have these (ϕ', c′) values as the median values if, within the range of p′ considered, the number of test points above this design line is equal (or nearly equal) to the number of test points below it.

The above new method is most suitable for application of "big data" analysis. By inputting the coordinates of the test results as well as those of a number of probable design median (ϕ', c′) lines into the computor, the true median values can be determined with successive trial of different deign lines. It sould be noted that different sets of (ϕ', c′) values are to be assigned to different types of soil. Different places might require different sets.

Nevertheless, before "big data" can be applied and when only a limited amount of data is involved, the manual fitting method as proposed by Wong (2020b) can be adopted.

There are certain distinct advantages of this new method:

(1) It is simple to use as no advance statistical analysis is involved and it also avoids the problem of the results being affected by a few extraordinarily high values.

(2) The triaxial test results together with the design envelope provide a very good visual view and hence more engineering judgement can be carried out. Moreover, if a conservative design is

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necessary, the number of test points above the proposed design line can be increased to say 60 % or more of the total number.

(3) It takes into account the effect of ground condition on shear strength by introducing the p′ term as one of the soil shear strength parameters. Only with p′ and (ϕ', c') together, it is possible to estimate the soil shear strength at any site location, as discussed in details in Wong (2020b). The above paper also point out a very vital issue that (ϕ', c') are just shear strength parameters. Accordingly, any location with a lower p′ but higher (ϕ', c') does not necessarily have a higher shear strength than the location with higher p′ but lower (ϕ', c').

3.2.2 Determination by in-Situ Testing

To determine the soil shear strength in-situ will be much more involved. In theory according to Table 1, CPT, plate loading and pressuremeter can yield drained values (ϕ' and c') as well as undrained values. However in practice, measurement and interpretation of in-situ results will be extremely difficult, if not impossible. Moreover, some of these tests have high operating cost (e.g. plate loading), some difficult to mobilize to site (e.g. CPT) and some difficult to perform (e.g. pressuremeter)

An obvious choice is the SPT as this test is not only simple to operate in practically all soil types both above and below groundwater table. Moreover, the equipment required is relatively simple, rugged and permits frequent test. Lastly but not least, SPT can be correlated to the majority of the engineering properties, including shear strength and deformation characteristics.

Table 2 (which is basically reproduced from Table 6 and 7 of Wong 2020b) summarizes the effect of soil type on SPT, dry density and shear strength parameters. This table also demonstrates that with the same increase in dry density, weathered soil has a much larger increase in shear strength than fill/colluvium. Moreover, for SPT value of the same order, the average shear strength is also of the same order, irrespective of the type of soil, which in this case is weathered soils and fill/colluvium.

Table 2. Variation of average soil shear strength with SPT for various soil types based on median (ϕ', c') values from Table 6 of Wong (2020b)

Soil type	SPT (In-situ dry density, Mg/m^3)	Median (ϕ′, c') values (ϕ' in degree & c' in kPa)	*Average soil shear strength kPa
Fill/ colluvium	3 – 6 (1.20 – 1.40)	34, 3	87.31
	6 – 12 (1.40 – 1.60)	35, 4	91.52
	12 – 24 (1.60 – 1.75)	36, 5	95.81
	24 – 42 (1.75 – 2.00)	38, 7	104.67
C/HDG	8 – 20 (1.20 – 1.40)	36, 5	95.82
	20 – 50 (1.40 – 1.60)	38.5, 6.5	105.93
	50 – 100 (1.60 – 1.75)	42, 9	121.35
	100 – 200 (1.75 – 1.90)	45, 10	135.00

* Average soil shear strength = Average mean effective normal stress x tan ϕ′ + c'

Average mean effective normal stress for p′ ranging from 0 to 250 MPa = (250 + 0)/2 = 125 MPa ,

(ϕ′, c') are median (ϕ', c') values for various SPT ranges as given in Table 6 of Wong (2020b)

3.3 Soil Compressibility

3.3.1 Types of Compressibility Parameter and their Respective Definition

Soil compressibility parameters are normally defined in accordance with the testing conditions and there are the following three major types (see Head, 1986):

(1) Anisotropic stress condition defining the Young's modulus (E). This is commonly referred to as elastic modulus. In this anisotropic case E is just defined as

$E = (\delta\sigma_Z - \delta\sigma_C)/\delta\varepsilon_Z$ with σ_Z = Vertical stress

σ_C = All round lateral stress

$\delta\varepsilon_Z = \delta z/z$

z = Depth of specimen

or $E = \delta\sigma_Z/\delta\varepsilon_Z$ when $\delta\sigma_C = 0$

(2) Isotropic stress condition defining the bulk modulus (B). In this isotropic case in which an all round equal pressure δp is applied, B is defined as

$B = \delta p/\delta\varepsilon_V$ with $\delta\varepsilon_V = \delta V/V$

V = Volume of specimen

In soils, however, it is the reciprocal of B that is commonly used,

$m_v = 1/B = \delta\varepsilon_V/\delta p$

and m_V is termed as coefficient of volume compressibility.

(3) Rigid lateral boundary condition defining the constrained modulus (D). In this case, only a vertical stress is applied and the lateral boundary is so rigid that there is no lateral deformation, thus yielding

$D = \delta p/(\delta h/h)$ with h = depth of specimen

In soils, it is again the reciprocal of D that is commonly used, and this is also termed as coefficient of volume compressibility

$m_V = 1/D$

Nevertheless, it should be noted that with the lateral boundary fixed in this case, the vertical stress (p) applied in the later stages will be very high. Accordingly, a semi-log plot of void ratio (e) versus $\log_{10}$ p is normally adopted so that a new compressibility parameter termed: Compression Index, C_c, is introduced and this is defined as

$C_c = \delta e/\delta \log_{10} p$

In spite of the fact that 1/B and 1/D are both termed as coefficient of volume compressibility, it should be noted that these two terms are quite different as the testing conditions are also quite different. The relationship between the various m_v compressibility parameters so far introduced has been further discussed in Wong (2020a).

3.3.2 Practical Considerations and Applications

So far the following compressibility parameters: E, B, m_v, D & C_c have been introduced and these are as summarised in Table.3.

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Table 3. Compressibility parameters under different testing conditions

Testing Condition	**Compressibility Parameter**	**Assumptions Required**	**Applications**	**Testing method**	
Anisotropic stress Condition	Elastic modulus, $E = \delta\sigma_Z/\delta\varepsilon_z$	Linear elastic normally required	Cases in which loading area small comparing with thickness of compressible layer	Triaxial consolidated compression testing, either one of the following: CQP_M CQP_S CD_M CD_S	Shearing stage
Isotropic stress condition	Bulk modulus $B = \delta p/(\delta V/V)$	Not necessary required to be linear elastic	Practically no application in practice		Triaxial consolidation Stage
	Coefficient of volume compressibility $m_v = 1/B$				
Rigid lateral boundary condition	Constrained modulus $D = \delta p/(\delta h/h)$	Not necessary required to be linear elastic	Cases in which loading area large comparing with thickness of compressible layer	Oedometer testing	
	Coefficient of volume compressibility $m_v = 1/D$				
	Compression index $C_c = \delta e/\delta \log_{10} p$				

Moreover, it can be seen that these parameters can be measured in practice as follows:

(1) E is equal to the slope of the deviator stress versus axial strain curve in the shearing stage of the triaxial consolidated compression test, either the tangent modulus (E_t) or secant modulus can be determined. In this case, the anisotropic stress condition applies.

(2) B can be determined from the triaxial consolidation stage of triaxial consolidated compression test and this is an isotropic stress condition, m_V being the slope of the volumetric strain versus consolidation pressure curve. Nevertheless, such loading condition can rarely happen in practice.

(3) D (or m_v) is equal to the slope of the vertical strain versus p curve and C_c is equal to the slope of the void ratio versus $\log_{10}$ p curve. It should be noted that both C_c and m_v have been determined from the same test and the difference is only because of the method of calculation. Moreover, C_c and m_v are not necessarily for linear elastic materials.

(4) As for in-situ testing, the most useful one is SPT. The other in-situ tests as in Table 1 in theory can provide the information for compressibility. However, those data have not yet been well documented, thus limiting their application. In practice, it is obvious from Table 3 that only the two parameters (E and C_c) are useful for engineering design, normally E for granular soils and C_c for the more cohesive ones. Moreover, the assumptions required and testing restraints for E and SPT are basically similar.

Accordingly, E can be assumed to be proportional to SPT. In fact, this is the current Hong Kong empirical design practice of assuming E (in Mpa) equal to SPT value. With consistency being an indication of its compressibility, the above assumption can be further justified in Table 4 which summarizes the classification of transported and weathered soils of different consistency.

Table 4. Classification of transported and weathered soils in accordance to SPT (based on Table 4 of Wong, 2020b)

Dry density	**Transported soil**			**Weathered soil**		
	Soil Consistency	Soil type [(1)]	SPT [(2)]	Soil consistency	Weathered state	SPT [(2)]
< 0.8	Very soft	MC	$N \leq 1$			
0.8 – 1.2	Soft to very soft	MC, M-sandy clay M-clayey sand	$1 < N \leq 3$	--	--	--
1.2 – 1.4	Medium firm or loose	ALL-C, MS, F ALL-Sandy clay	$3 < N \leq 6$	Very loose to loose	Grade VI to Grade V	$8 < N \leq 20$
1.4 – 1.6	Stiff to very stiff or medium dense	ALL-C, MS ALL-Clayey sand, F, Re coll, CF	$6 < N \leq 12$	Medium dense	Grade V	$20 < N \leq 50$
1.6 – 1.75			$12 < N \leq 24$	Dense	Grade V to Grade IV	$50 < N \leq 100$
1.75 – 2.0	Hard or dense	Old coll, MS, ALL-sand, CF	$24 < N \leq 42$	Very dense	Grade IV	$100 < N \leq 200$[(3)]
> 2.0	Very dense	CF	$42 < N$			

Notes: 1. MC = Marine clay, M-sandy clay = Marine sandy clay, M-clayey sand – Marine clayey sand, MS = Marine sand; ALL-C = Alluvial clay, ALL-sandy clay = Alluvial sandy clay, ALL-clayey sand = Alluvial clayey sand, ALL-S = Alluvial sand; F = Normal filling without compaction except by its own weight, CF = Compacted fill; Re coll = Recent colluvium, Old coll = Old (or ancient) colluvium

2. SPT value corrected as in Skempton (1986).
3. Max SPT measurable is normally limited to 200.

The following special features can be observed:

(1) Firstly all transported soils (fill/colluvium in both areas, MD and ALL) can be further sub-divided as in Note (1) of Table 4.

(2) Weathered soils comprise mostly C/HDG and C/HDV and their respective classification is based only on the weathering state as the difference in engineering properties between these soils is not significant.

(3) The SPT value provides a more quantitative indication of the soil compressibility than such descriptive terms as loose, dense and hard, etc. From this table, SPT is related to compressibility.

As for C_c, this in theory can be applied to both granular and cohesive materials when the loading area is large in comparison with the thickness of the compressible layer. However, in practice, this is normally only applied to the more cohesive materials. From a detailed study of the marine and alluvial deposits in Hong Kong, Wong (1993) reckons that as far as Hong Kong soils are concerned, there is no correlation whatsoever between C_c and Atterberg limits. On the other hand, C_c is closely related to a parameter: corrected initial void ratio ($\mathbf{\hat{e}_o}$), which is obtained by projecting the loading $\log_{10}$ p versus

e_o curve to cut the vertical line p = 10 kPa, the ordinate of this intersection point being the $\hat{\mathbf{e}}_{\mathbf{o}}$. Moreover, $\hat{\mathbf{e}}_{\mathbf{o}}$. can be estimated from e_o which can be quite simply calculated from a knowledge of water content (w) and particle density (G) of MD or ALL. Accordingly, the compression index (C_c) for these soils can be easily estimated.

3.4 Soil Conductivity

3.4.1 Theoretical Background

There are two types of hydraulic properties: coefficient of consolidation (c_v) and permeability (k). These can be measured indirectly at the same time in the laboratory either by oedometer testing or the consolidation stage of a triaxial consolidated compression test. The major difference between these two parameters is that c_v is a lumped parameter introduced by Terzaghi (1943) in deriving his one-dimensional consolidation theory whilst k is a parameter measuring directly the soil resistance to fluid (usually water) flow.

Theoretically speaking k is defined from the Law of Darcy (1856) for laminar flow, applying to any viscous fluid flow through any material under any temperature. This is related to an intrinsic (or absolute or specific) permeability K (in m^2) usually adopted in such fields as soil science and agriculture as follows:

$$k = K g \gamma_w / \mu$$

where g = Acceleration due to gravity (m/s^2)

γ_w = Water unit weight (kN/m^3)

μ = Dynamic viscosity ($kN\ m^{-1}s^{-1}$)

For most practical engineering problems, the definition adopted by Tschebotarioff (1973) is most useful: k of a soil is defined as the imaginarily average velocity of flow which will occur under the action of a hydraulic gradient of unity through the total cross-sectional area (voids + solids) of the soil..

On the other hand, c_v is just a derived parameter calculated from Terzaghi's consolidation equation:

$$k = (\gamma_w)\, c_v\, m_v$$

with c_v = Coefficient of consolidation (m^2/s)

m_v = Soil compressibility (m^2/kN).

It should be noted that quite unlike c_v, k can also be measured directly in the laboratory as well as in-situ, both being based on Darcy's Law. The equations for calculating k is fairly simple and detailed description being given in Section 10 of Head (1986). In case of in-situ testing, as the normal practice is to measure from a porous section in a vertical borehole, the measured k value will be affected quite considerably by the size and location of this porous section relative to the various soil/rock strata as well as the direction of flow.

3.5 Testing Results

The test results processed in the past years are as indicated in Tables 5 and 6, with Table 5 summarizing the c_v and k results for various major soil types for IC testing and Table 6 for in-situ k values. It should be noted that the measurement of c_v in-situ is difficult, if not impossible. The same applies to measurement of k in-situ for MD and ALL as the testing time will be extremely long in view of their very low permeability.

Table 5. Consolidation coefficient, c_v and permeability, k determined by IC (isotropic consolidation in a triaxial cell) for various soil types

Soil type		Coefficient of consolidation, c_v (m^2/year)				Permeability, k (m/s)			
		No of test Results (1)	$(c_v)_{50}$	$(c_v)_{10}$ – $(c_v)_{90}$	(2)Dev c_v	No of test Results (1)	$(k)_{50}$	$(k)_{10}$ – $(k)_{90}$	(2)Dev k
C/HDG		509	13400	1160 – 43000	1.569	509	9.25×10^{-7}	8.70×10^{-8}– 3.58×10^{-6}	1.614
C/HDV		375	8450	614 – 39700	1.811	337	4.10×10^{-7}	5.10×10^{-8}– 2.45×10^{-6}	1.682
Fill/Coll Granitic areas		228	2399	111 – 30200	2.434	227	3.92×10^{-7}	1.64×10^{-8}– 3.29×10^{-6}	2.301
Fill/Coll Volcanic areas		159	2380	95 – 37000	2.590	185	2.94×10^{-7}	9.6510^{-9}– 3.55×10^{-6}	2.565
MD	F > 20%	114	0.910	0.238 – 3.780	1.200	74	1.82×10^{-10}	2.18×10^{-11} 8.20×10^{-10}	1.593
ALL		104	1.220	0.470 – 9.700	1.315	70	9.05×10^{-11}	2.28×10^{-11}– 6.35×10^{-10}	1,445
MD	F < 20%	24	3680	71.0 – 20000	2.450	18	1.19×10^{-7}	8.65×10^{-9} 7.35×10^{-7}	1.929
ALL		(too few test results for meaningful analysis)							

Notes: 1. Test results being from single & multi-stage triaxial testing, each stage representing one test result
2. Dev (c_v) = $\log_{10}\{(c_v)_{90}/(c_v)_{10}\}$ and Dev (k) = $\log_{10}\{(k)_{90}/(k)_{10}\}$

Table 6. Variation of median k, interdecile range of k and its deviation with SPT for various soil types as determined by various types of in-situ testing

Soil type	(1) No of test results	SPT range	Median k (m/s)	Interdecile range of k (m/s)	(2)Dev k
C/HDG	12 (10 FH + 2 RH)	< 20	8.50×10^{-7}	2.24×10^{-7}– 2.76×10^{-5}	2.091
	38 (30 FH + 8 RH)	20 – 50	1.15×10^{-6}	1.33×10^{-7}– 2.98×10^{-5}	2.350
	20 (20 FH + 3 RH)	50 – 100	2.00×10^{-6}	3.82×10^{-7}– 2.00×10^{-5}	1.719
	33 (24 FH + 9 RH)	> 100	1.15×10^{-6}	1.72×10^{-7}– 1.66×10^{-5}	1.985
	24 FH	> 100	1.00×10^{-6}	1.33×10^{-7}– 9.20×10^{-6}	1.840
	9 RH	> 100	1.12×10^{-6}	3.01×10^{-7}– 2.11×10^{-5}	1.846
	103 (81 FH + 22 RH)	< 20 to > 100	1.12×10^{-6}	1.88×10^{-7}– 1.92×10^{-5}	2.028
	81 FH	< 20 to > 100	1.02×10^{-6}	1.50×10^{-7}– 1.24×10^{-5}	1.917
	22 RH	< 20 to > 100	1.49×10^{-6}	3.94×10^{-7}– 3.50×10^{-5}	1.949
C/HDV	10 FH	< 20 to < 50	7.62×10^{-7}	5.00×10^{-8}– 2.00×10^{-6}	1.602
	18 FH	50 to > 100	1.33×10^{-6}	1.76×10^{-7}– 3.84×10^{-5}	2.339
	28 FH	< 20 to > 100	1.00×10^{-6}	8.80×10^{-8}– 3.65×10^{-6}	1.618
Fill/ colluvium	9 (8 FH + 1 RH)	< 6	5.62×10^{-6}	4.62×10^{-7}– 2.22×10^{-5}	1.682
	10 (7 FH + 2 RH+ 1 CH)	6 to 24	6.35×10^{-6}	3.20×10^{-7}– 5.00×10^{-5}	2.194
	22 (17FH+ 3 RH+ 2 CH)	<6 to > 24	5.00×10^{-6}	3.34×10^{-7}– 2.93×10^{-5}	1.943

Notes: 1. No. of test results is the summation of the following different test types
FH – Falling Head test, RH – Rising Head test and CH – Constant Head test
2. Dev (k) = $\log_{10}\{k_{90}/k_{10}\}$

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Moreover, the following special features can be observed from these test results. Firstly, soil type has a dominant effect on the c_v and k values. In moving from MD and ALL to weathered soils, the increase in c_v and k can be up to ten thousand times or more. This is considerably larger than the variation within the same soil type as indicated by their respective dev (c_v) and dev (k) values.

It should be noted that as oedomer testing tends to under-estimate the c_v and k results for the more granular soils (see Wong, 2017), it follows that such results should be excluded in the future big data analysis.

4 Methods Of Data Processing for Application of "Big Data"

4.1 General Considerations

As far as soil engineering properties are concerned, it is soil type classified in accordance to the formation process that is most vital. According to this classification, there are only three groups: (1) weathered soil (mostly C/HDG or C/HDV), (2) fill/colluvium in granitic or volcanic areas and (3) MD or ALL. This is quite different from classical soil mechanics in which fines content (F) and in-situ dry density (τd) are the controlling parameters affecting the soil engineering properties. As demonstrated by the test results in Wong (2020a), two soil types with similar F and τd can have their cv and k values differing by more than thousand times. The same applies to the SPT values, but only at a lesser degree as demonstrated by Table 4.

4.2 Specific Guidelines

In order that the data can be useful for big data applications, the following major guidelines have to be followed in the processing works.

Firstly, for every set of data with engineering properties, the soil type must be firstly identified as there is usually a tremendous difference between various soil types, in particular the conductivity parameters. For future use, soil type should be entered in the more specific pattern as follows;

(1) Weathered soil – C//HDG; Weathered soil – C//HDV; Weathered soil – (other weathered soil, e.g. granodiorite).
(2) Fill/colluvium – granitic area; Fill/colluvium – volcanic area; Fill/colluvium – (other unidentified areas).
(3) MD – Marine clay; MD – Marine sandy clay; MD – Marine clayey sand; MD – Marine sand.
(4) ALL – Alluvial clay; ALL – Alluvial sandy clay; ALL –Alluvial clayey sand; ALL – Alluvial sand.

Secondly, SPT might not be present in every set of soil data. This should be entered together with soil type, overburden depth and dry density range (should there be any). The location where each sample was taken should also be entered as this basically serves as a more secondary role to define the soil type and is useful when soil type was not identified in the original description.

Finally, other more secondary data to be entered comprises the following items: sampling date (d/m/y), testing date (d/m/y), sample depth (m), depth at which sample tested (m), etc,

4.3 Data Entering for Soil Shear Strength Determination

As proposed in the recent paper by Wong (2020b), shear strength can be determined from either the data of (1) triaxial testing or (2) in-situ testing or by both. For data entering, there are two major types of data: (1) primary data and (2) secondary data.

(1) Primary data

For triaxial testing, this is q versus p' in $CQP_M/CQP_S/CD_M/CD_S$ test at the critical state, in which q = (σ_1 - σ_3)/2, p' = (σ_1' +σ_2' +σ_3')/3 or (σ_1' + σ_3')/2 and the critical state is at which (σ_1' /σ_3') equal to a maximum. For multi-stage testing (CQP_M and CD_M), the specific stage number must be identified. In case of SPT, this must be identified if it is either below or above an in-situ sample in which triaxial testing has been carried out.

(2) Secondary data

Each set of primary data must be accompanied by the secondary data listed as follows. Secondary data are basically for identifying the respective samples in which shear strength testing (triaxial or SPT) has been carried out. These should be in the following sequence: Sample location, Date of testing, Hole No, Sample No, Sample depth, Fines content, In-situ dry density, testing laboratory and any other data relevant to the specific data set.

4.4 Data Entering for Soil Compressibility Determination

In case of in-situ testing for each soil, collect the SPT and the τ_d value (should there be any). As a preliminary design the elastic modulus is assumed to be directly proportional to the SPT. In case of laboratory testing, it is only E and C_c that will be useful in practical engineering design. The B value will not be considered as an isotropic case has no practical application. As for the D value, this derives from the same set of test results as C_c. However, the latter is more adaptable to mathematical evaluation. As in soil shear strength, the primary data and secondary data have to be entered. For E determination, enter the set of ($\sigma_Z - \sigma_C$) versus ε_Z values for each σ_C value together with the F and τ_d values. For each set of data, the average E as in Section 3.3.1 (1) together with σ_C must be entered

To determine C_c, the void ratio (e_o) versus $\log_{10}p$ values have to be entered for each oedometer test. In addition, the F and e_o of the in-situ sample have also to be entered. An alternative approach when no ooedomster testing is available is to estimate the corrected initial void ratio ($\hat{e}_o$.) from the water content (w) and particle density (G) values as proposed by Wong (1993). By assuming $\hat{e}_o$ to be approximately equal to e_o, a correlation between C_c and $\hat{e}_o$ can be established from the empirical relationship proposed in the above paper: for MD, $C_c = 0.39(\hat{e}_o - 0.4)$ and for ALL, $C_c = 0.325(\hat{e}_o - 0.3)$.

4.5 Data Entering for Soil Conductivity

The most vital data to enter is soil type, then fines content (F) and finally in-situ dry density (τ_d). Unlike shear strength and compressibility, SPT is basically not required. A very special feature is that k for engineering design is normally measured in-situ. Moreover, for each soil type c_v and k appear to be not related to such parameters as F and τ_d , except in the case of MD and ALL. For these two soil types, c_v and k increase quite considerably for F < 20%. In entering in-situ k values, the types of additional data to be entered comprise: location and depth of testing, testing type (FH/RH/CH), soil layering (if any) and other special features relating to the particular site.

One final vital point to note is that for design in soil types other than MD/ALL, it is normally the design for drainage in which only the in-situ k is required. The design for consolidation settlement as in MD/ALL is not applicable. In the latter case, it is the c_v value that is controlling.

5 Methods of Determining Soil Design Parameters

5.1 General Considerations

In order to evaluate the required design parameters (e.g., ϕ', c') from the original data processed, certain programmes are required to be keyed in first, as listed below. Moreover, these are required to be updated from time to time with the rapid development of modern technology.

5.2 Shear Strength Design Parameters

The following two programmes are required: (1) for determining the (ϕ', c') from the p' – q data for p' within a certain range and (2) for correlating between average soil shear strength and SPT for a certain range of p'.

5.3 Compressibility Parameters

Programmes are required for determining E and C_c To determine E, a programme for evaluating E from a set of ($\sigma_Z - \sigma_C$) versus ε_Z values within a certain range of σ_C, and another one for correlating the E value thus obtained with SPT. For C_c, a programme based on the paper by Wong (1993) is required to calculate the C_c and hence the (**$\hat{e}_o$**) value for each set of oedometer testing. In case oedometer testing is not available, the programme can also provide an estimate of the (**$\hat{e}_o$**) value from the water content and particle density of the soil.

5.4 Conductivity parameters

In considering the conductivity parameters, a programme is required to be keyed in so as to list the c_v, k as well as the F and τ_d values of laboratory testing carried out within a certain specified area. The median and interdecile range of c_v and k values are then evaluated for each soil type within a certain range of F and τ_d values. The same operation can be applied for in-situ k values.

Nevertheless, the following special features should be taken into consideration. Firstly, because of the very permeable nature, c_v is practically not required for design for weathered soils and fill/colluvium. On the other hand, the very impermeable nature of MD/ALL also render their k values not useful in design. Secondly, the k value for design must be determined in-situ as the presence of local voids cannot be reproduced in the laboratory and these would have tremendous effect on the k value. Accordingly, the median and interdecile range of in-situ k values thus obtained are for reference only. Finally, except for MD/ALL with F < 20%, the effect of such secondary parameters as F and dry density is relatively insignificant in comparison with the effect of soil type.

6 Conclusions

The data processed in the past few decades has demonstrated quite conclusively that the major engineering properties are dominated by two primary parameters: soil type and SPT, in particular soil type classified in accordance to the formation process. With this classification, Hong Kong soils are divided into three major groups: weathered soil, fill/colluvium and MD/ALL. Another important parameter is SPT.

The relative importance of these two parameters does vary. There is a tremendous increase up to ten thousand times or more for conductivity in moving from MD/ALL to weathered soils, but the effect of SPT is not to such an extent. For shear strength and compressibility, the effect of these two parameters is somewhat less significant. In both cases, the effect of in-situ dry density (τ_d) also plays a fairly important secondary role. Within each soil type, shear strength will increase but compressibility decrease with the increase in τ_d. However, in moving from one soil type to another, the above trend no longer holds. For example, as indicated in Table 2, in moving from fill/volluvium to weathered soil, a higher τ_d sample in the former group might not necessarily have a higher shear strength than the one with a lower τ_d in the latter group. In general, the correlation between SPT and shear strength or as in Table 4 between SPT and compressibility is irrespective of soil type. It should be noted that as far as Hong Kong soils are concerned, this type of soil is weathered soils and fill/colluvium for shear strength and this includes MD/ALL for compressibility.

So far this paper has only outlined the basic principles for application of "big data" by identifying the major soil engineering properties for design and the major primary and secondary parameters affecting them. In order to ensure the usefulness of such data, this must be re-processed in a more orderly manner as proposed. Moreover, to obtain the required parameters for design, programmes must be keyed in to perform such operations.

Finally it must be pointed out that this paper has been dealing exclusively with soil materials (weathering state Grade IV to VI as in Table 4), not rock materials (weathering state Grade I to III). Nevertheless, as far as normal civil engineering works are concerned, it is the conductivity parameter that is most vital, then shear shear strength in case of rock slope and tunnelling. Moreover, laboratory testing is usually not required.

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