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SUMMARY The use of pressuremeter to measure the deformation characteristics of completely weathered granite in Hong Kong for the design of substructures are described in this paper. Correlation between pressuremeter moduli and limit pressures from pressuremeter tests and N-values from standard penetration tests are made. Bearing pressure calculated from pressuremeter modulus is compared with those computed with N-values. Values of coefficient of modulus of weathered granite for calculating lateral resistance of deep foundations are reported.

INTRODUCTION

High-rise buildings constructed on hillslopes are very common forms of structure in Hong Kong. Deep foundations such as hand-dug caissons are normally employed to transmit building load and lateral load from wind force or soil pressure to suitable subsurface strata. For caissons bearing on residual soils, the determination of their bearing capacity normally follows the empirical solution developed by Terzaghi & Peck (1967). The lateral resistance capacity of caissons is normally based on the concept, derived by Matlock & Reese (1960) and Davisson & Gill (1963), of infinite elastic beam embedded in an elastic medium.

The pressuremeter test, which measures in-situ the lateral deformation characteristics of ground at a particular depth, is used in predicting the bearing and lateral resistance capacity of deep foundations. The pressuremeter modulus, from which the coefficient of subgrade reaction can be derived and the settlement beneath a foundation can be assessed, and the limit pressure, from which the ultimate bearing capacity of a foundation can be calculated, were determined from the pressuremeter test. Relationship between the results of pressuremeter tests and the commonly used standard penetration tests are discussed along with the variation of pressuremeter data with depth.

PRESSUREMETER METHOD

The apparatus used is a Menard GA type Pressuremeter. The pressuremeter consists of a radially expandable cylindrical probe with three independent cells connected to a monitoring unit. The central cell is a measuring cell and is flanked on top and bottom by two guard cells. The measuring cell is pressurized by water through a pneumatic control in the monitoring unit and the guard cells are inflated by gas. The test is performed by inserting the probe into a 60 mm diameter dry-drilled hole and the volumetric displacement of the probe is measured with applied pressure and time.

A typical pressuremeter test result is shown in Figure 1. The creep pressure, P_f, corresponds to the beginning of soil failure shown by the drastic increase in volume change with pressure. The limit pressure, P_l, corresponds to the limiting state of failure of soil as shown by the abscissa of the asymptote to the pressuremeter curve. Based on the straight line portion (pseudo-elastic phase) of the pressuremeter curve, the Pressuremeter Modulus, E_M, is defined by Centre d'Etudes Menard (1975) as

$$E_M = 2(1+V)\ V_M \frac{\Delta P}{\Delta V}$$

where E_M = Pressuremeter modulus

V = poisson's ratio

V_M = dimensional coefficient of probe

$\frac{\Delta P}{\Delta V}$ = slope of pressuremeter curve in pseudo-elastic phase

The settlement beneath a deep foundation can be predicted using the semi-empirical formula proposed by Menard and Rousseau (1962) and later modified by Centre d'Etudes Menard (1975). The formula utilizes the concept that the total settlement is the sum of volumetric or consolidation and deviatoric or shear deformation. The calculation involves dividing the soil strata beneath the foundation into uniform layers. The bearing capacity of a foundation of a specified allowable settlement can be calculated based on the value of pressuremeter modulus determined by test.

The lateral resistance of a deep foundation, from the concept of an infinite beam embedded in an elastic medium, can be determined by solving the differential equation of

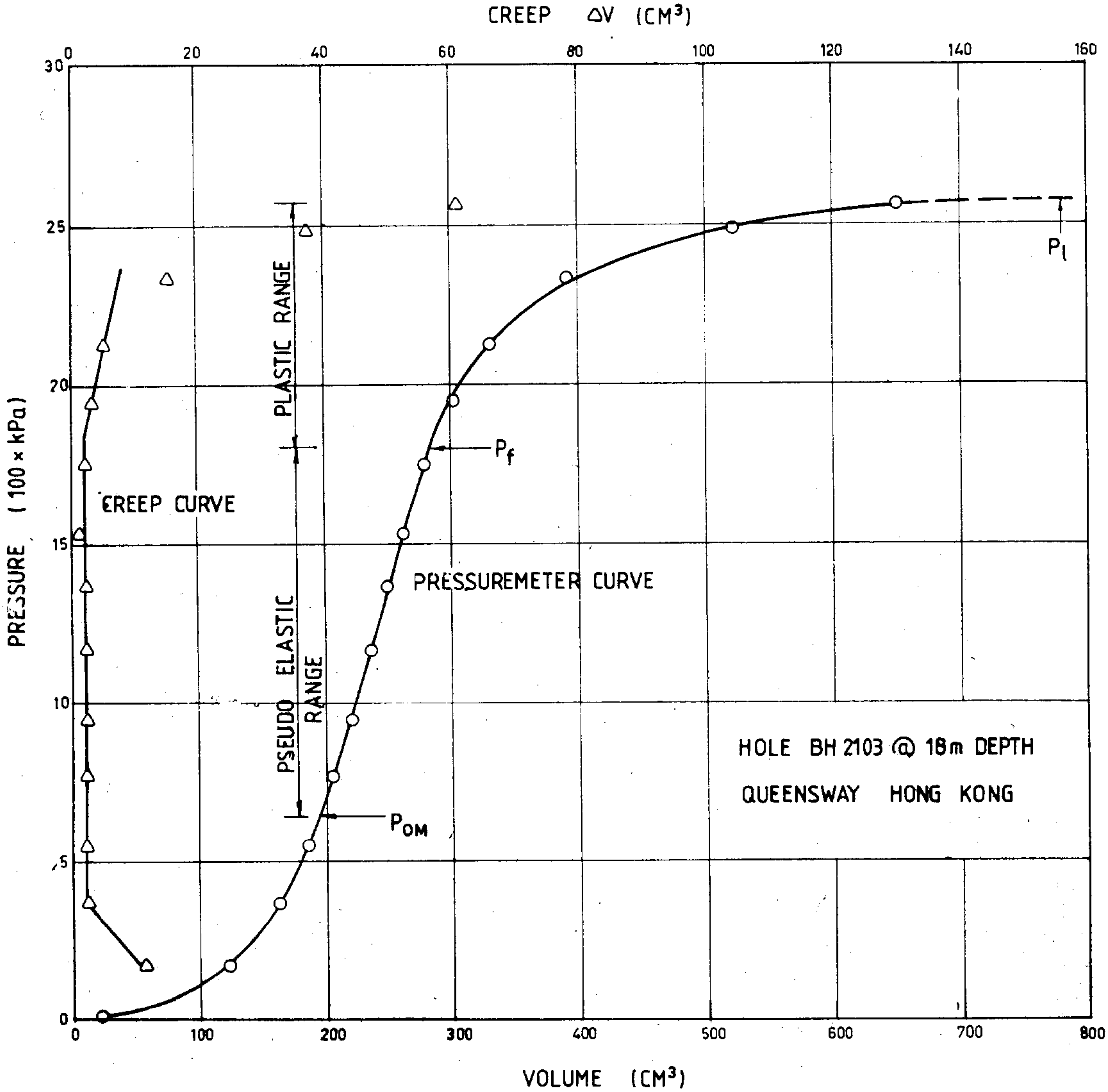


FIG. 1 TYPICAL PRESSUREMETER TEST RESULTS

$$\frac{d^4x}{dZ^4} = k_s . x$$

where Z = length of the caisson or pile

x = horizontal movement

k_s = modulus of horizontal subgrade

Solutions to the differential equation have been presented by Davisson & Gill (1963) for constant k_s and k_s in stepped variations, and, by Matlock & Reese (1960) and Davisson (1970) for k_s linearly increasing with depth. The modulus of horizontal subgrade, k_s, can be derived from the pressuremeter modulus by formula described in Centre d'Etudes Menard (1975).

GROUND CONDITIONS

The pressuremeter test data presented in this paper were from tests performed in completely weathered granite found mainly in the northern and central part of Hong Kong Island, and the southern part of Kowloon Penisula.

The Hong Kong Granite is normally composed of felspar, quartz and hornblende and is typically a two-felspar biotite of fairly coarse texture. The decomposition of the granite is mainly caused by partial alternation of the unstable felspar and biotite into kaolinitic clay minerals under the chemical action of water, oxygen and carbon dioxide. The plagioclase normally decomposes first, followed by orthoclase and biotite while the quartz remained unaltered (Lumb, 1962a). The rate of diffusion of water into the granite is normally fairly high, leading to great depth of decomposition up to 60 m. As the degree of weathering decreases with depth, the grading of the granitic soil changes from clayey sand near the ground surface through silty sand to coarse sand at greater depth. Descriptions on the general properties of the weathered granite in Hong Kong have been extensively described by Lumb (1962a, 1962b, 1965).

The general properties of the weathered granitic soil encountered in this series of pressuremeter tests are given in Figure 2. The N-values as determined from the Standard Penetration Tests increased from about 10 to 130 within the completely weathered zone tested.

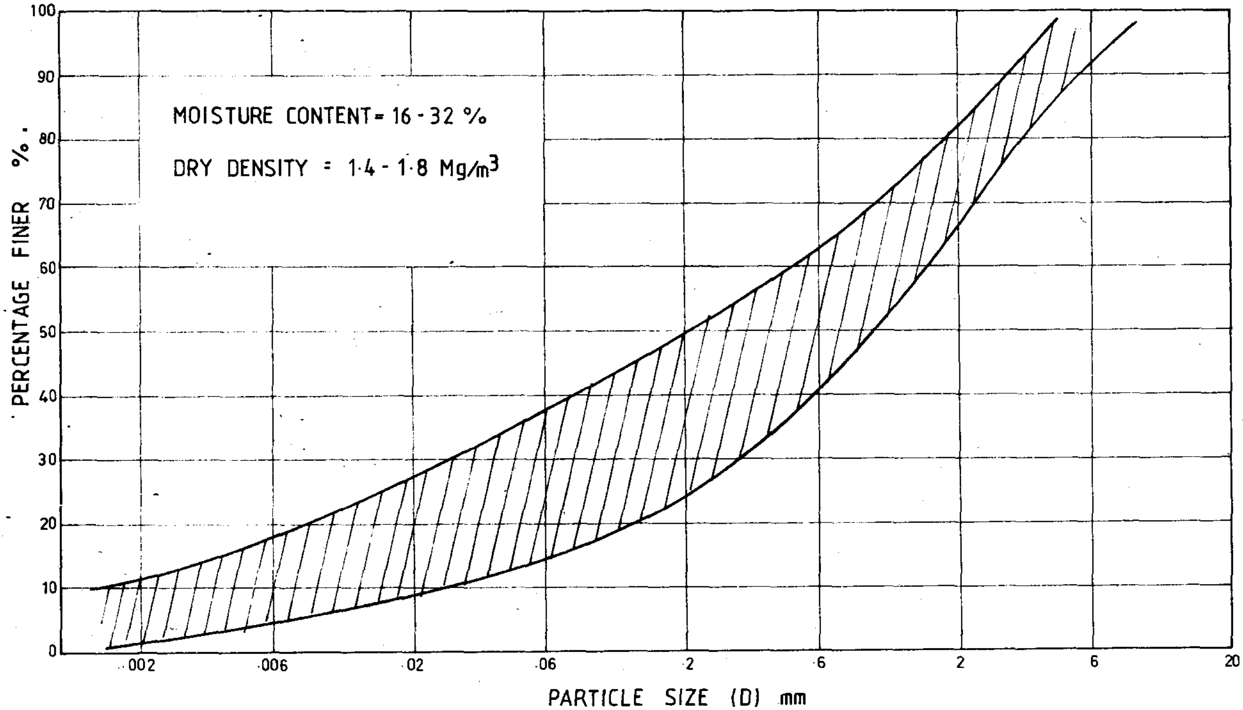


FIG. 2 PROPERTIES OF COMPLETELY WEATHERED GRANITE TESTED

DISCUSSION OF TEST RESULTS

The Standard Penetration Test (SPT), with its ease of performance and extensive correlation with parameters used in foundation design, is the prevalent method used in Hong Kong in evaluating the allowable bearing stress for foundation design. It is therefore logical to correlate the values of pressuremeter modulus, E_M, and limit pressure, P_l, determined from the Pressuremeter Tests with the N-values determined from the Standard Penetration Tests. A good correlation would allow the estimation of E_M values of soils from known N-values.

By plotting the data points determined in this series in the completely weathered granitic soil, a linear correlation was found between Log P_l and Log N and between Log E_M and Log N as shown in Figures 3 and 4, respectively. The linear coefficient determined from the least square method is 0.885 for P_l vs N (Figure 3) and 0.786 for E_M vs N (Figure 4). Similar correlation has also been obtained by Martin (1977) for residual soils derived from schists and gneisses of varying texture.

The pressure modulus E_M and the limit pressure P_l, like N-values, were also found to increase with depth. The rate of increase, however, cannot be clearly defined due to other factors such as groundwater condition, grain size distribution and mineralogy. A typical trend of the variation of E_M, P_l and N-values with depth is shown in Figure 5.

Values of coefficient of modulus variation, n_h, for calculating the lateral resistance capacity of caissons embedded in completely weathered granite, as determined from the pressuremeter tests, are given in Table 1.

TABLE 1 Coefficient of Modulus Variation

Relative Density	Medium Desne (N 50)	Dense (N 50)
Range of n_h	2 - 5	5 - 16
n_h (average)	3.8	7.8

Considering a circular-shaped deep foundation of 1.5 m diameter founded on a homogeneous completely weathered granite stratum, a comparison of the bearing pressures of the foundation computed from the pressuremeter method and other methods such as Terzaghi & Peck (1967), Meyerhof (1956) and Martin (1977) was made. As given in Table 2, a much higher value of bearing pressure was obtained from the pressuremeter method than the others.

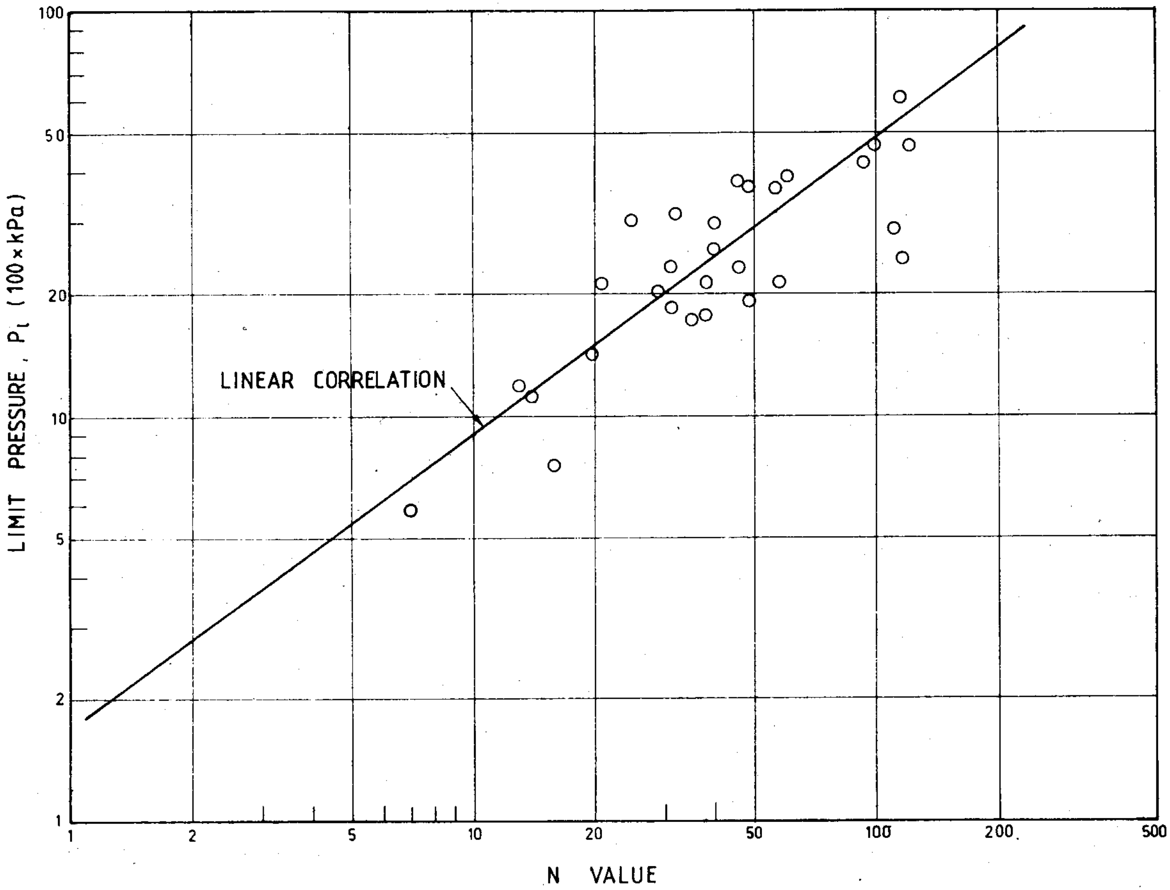


FIG. 3 VARIATION OF LIMIT PRESSURE VERSUS N-VALUS

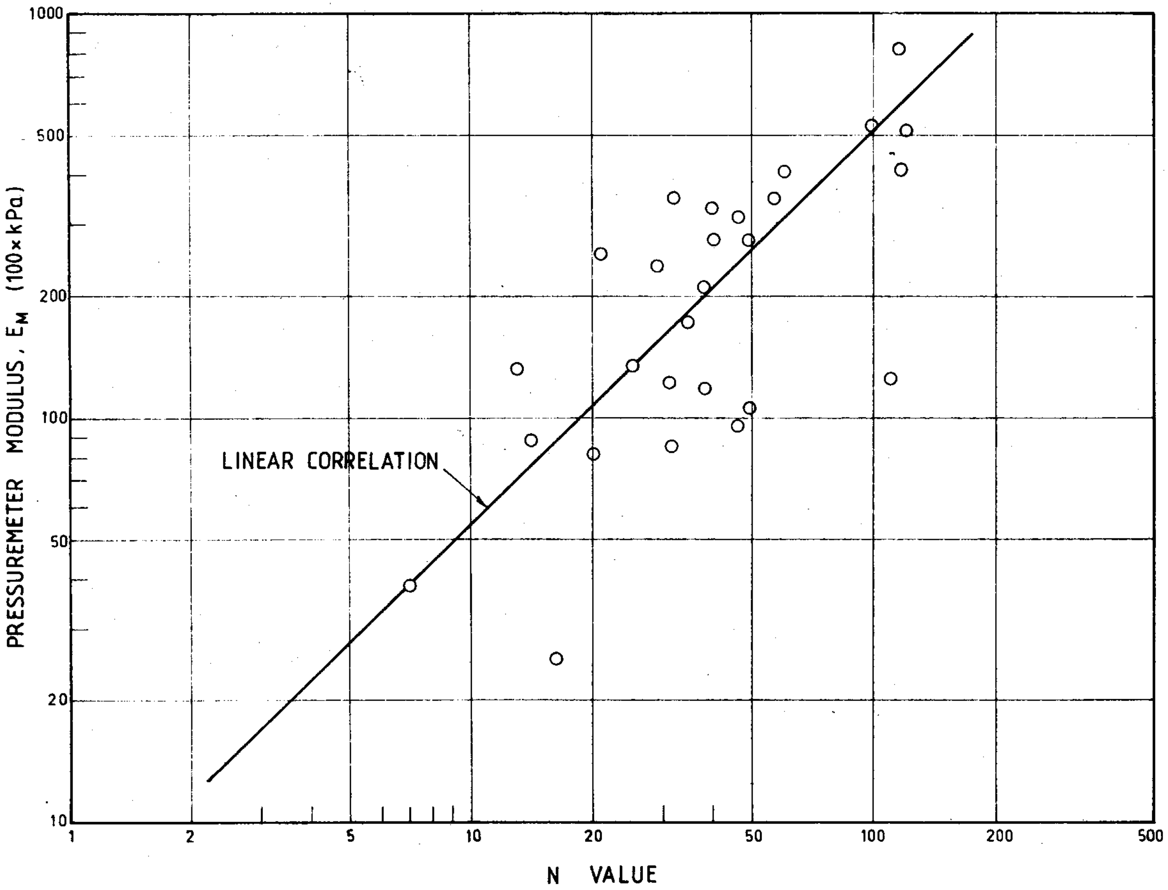
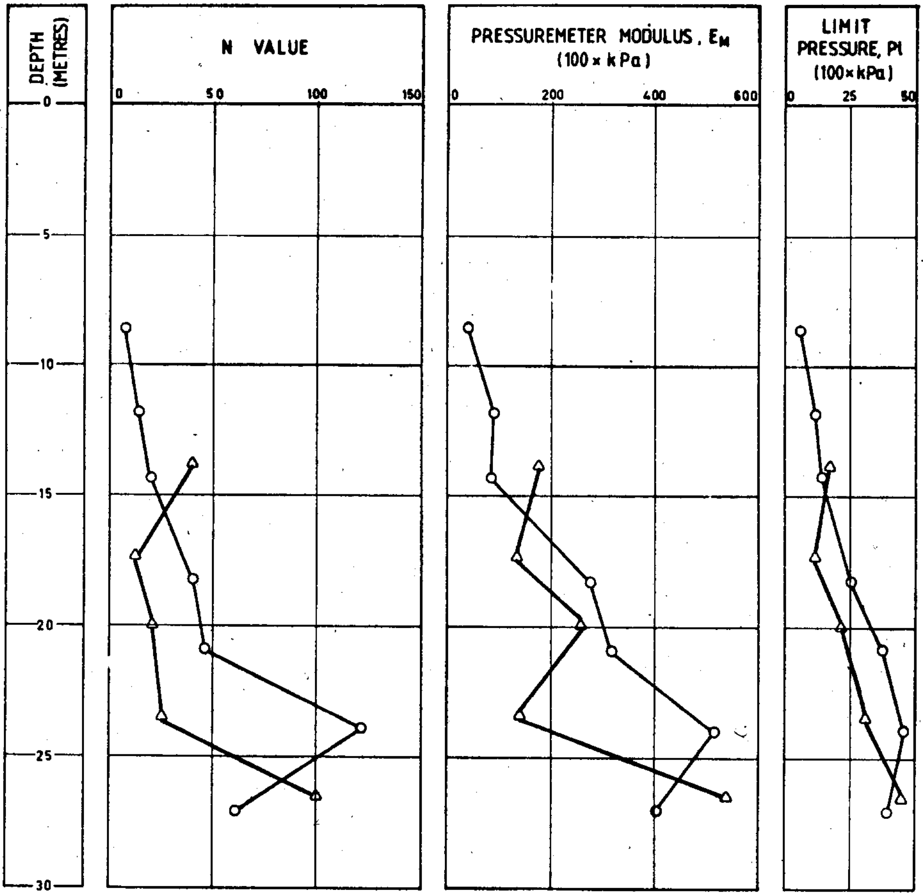


FIG. 4 VARIATION OF PRESSUREMETER MODULUS VERSUS N-VALUE



TEST LOCATION : o QUEENSWAY, HONG KONG
Δ SHAU KEI WAN, HONG KONG

FIG. 5 VARIATION OF PRESSUREMETER TEST RESULTS WITH DEPTH

TABLE 2 Comparison of Calculated Bearing Capacities

Method	Terzaghi & Peck (1948) Modified by Teng (1967)	Meyerhof (1956) Modified by Bowles (1974)	Martin (1977)	Menard & Rousseau (1962)
Bearing Capacity (N/m^2)	382	415	584	1123

Input data

Diameter of caisson, B	=	15. m
Depth of foundation, D	=	20 m
Allowable settlement, s.	=	13 mm
Reological factor, α	=	0.5
N-values from SPT, N	=	50
Pressuremeter modulus, E_M	=	26000 kPa
Bulk Density, γ	=	20 kN/m^3

CONCLUSIONS

The pressuremeter method allows inexpensive and rapid estimation of bearing capacity, settlement and soil modulus for the design of laterally loaded deep foundation such as caissons.

A linear logarithmic-logarithmic correlation has been shown to exist between the pressuremeter values (E_M, P_l) and the N-values from the Standard Penetration Tests. If pressuremeter tests are carried out in conjunction with the Standard Penetration Tests, bearing capacity and settlement of a deep foundation as well as its lateral loading capacity can be predicted. The pressuremeter method also seems to yield a higher bearing pressure than others.

The pressuremeter modulus and limit pressure determined from the pressuremeter test, as with N-values determined from the Standard Penetration Test, was found to increase with depth. The rate of increase however, cannot be clearly defined.

It is suggested that full scale loading tests should be carried out, in conjunction with the pressuremeter tests, to further verify the bearing capacity and lateral resistance of deep foundations.

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