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Interpretation of Static Cone Penetration Test with Triaxial Test to Determine Undrained Shear Strength of Clayey Soil

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Abstract. A series of static cone penetration tests and undrained triaxial tests are performed to determine undrained shear strength of clayey soil. The field static cone penetration test is conducted on Marine Clay Soil near Naygaon West, Mumbai. It will be very precise, if we able determine the undrained shear strength of clayey soil using field tests. Therefore in this research we have developed empirical relationship between the parameters of static cone penetration test and triaxial test, so that using the field parameters we can able to determine undrained shear strength of clay soil. Mainly the static cone penetration test gives end bearing, frictional resistance and corresponding cone factor of clay soil for various depths, which are then related with the laboratory parameters like Atterberg's Limits and Undrained Shear Strength of clay for the same depths. From the empirical relationship between field parameters and laboratory parameters we are able to develop coefficient called as cone factor K, which will help us to determine undrained shear strength of soil directly on site for similar types of clayey soil.

Keywords: Cone Penetration Test, Undrained Shear Strength, Cone Factor.

1 Introduction

1.1 A subsection sample

In geotechnical engineering practice most importance is given for determination of index properties and engineering properties of the soil sample. There are several laboratory and field methods are available for determination of these properties. In laboratory many of the tests are performed on disturbed and undisturbed soil sample. But these are the representative soil samples which gives the approximate values of the test results. There are very few tests which gives the results according to the field conditions. For major construction work it is very necessary to have a representative test data for further design process. Clay soil sample is very sensitive from construction point of view and it is very necessary to take care during construction.

In clay soil undrained shear strength parameter is very important from design perspective. So, it is very important to have representative values of undrained shear strength of clay soil sample. Laboratory triaxial test is one of the important test to determine undrained shear strength of clay soil and it gives representative results. But if we can able to find the undrained shear strength of soil directly on site then achieved values will be based on site conditions. In this research the correlation has been made between triaxial test and static cone penetration test to achieve the undrained shear strength values of clay soil directly on the site using static cone penetration test. Static cone penetration test can be used for wide range of geotechnical applications in clay soil.

2 Site Selection

The static cone penetration test can be performed on soft soil, so site is selected in such way that static cone penetration test can be carries out easily. The area around Naigaon West, Mumbai is deposited with marine clay. So, we have selected Naigaon West, Mumbai to carry out static cone penetration test. The site is located near Naigaon railway station, Mumbai which 5 km away from west coast of Arebian Sea.

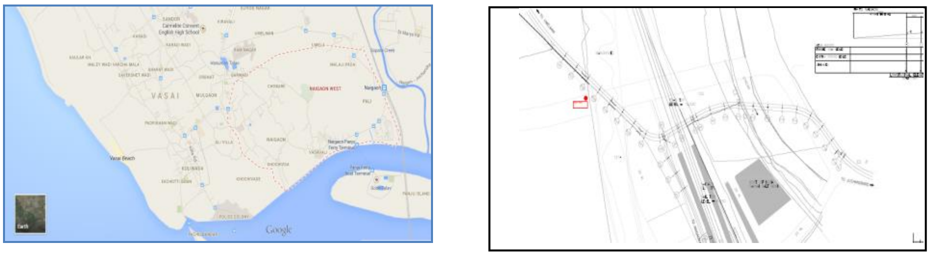


Fig. 1. Site location and map

The site map is shown below in Fig. 1 (Ref. Genstru Consultants Pune). On the same site Genstru Consultants were also performing geotechnical investigation for railway over bridge.

3 Field Testing and Sample Collection

3.1 Static cone penetration test : The static cone penetration test used to know soil strata and also to identify soil layers that may be problematic and require additional testing during investigation. The results of static cone penetration tests are used for geotechnical design process and various correlations between soil parameters.

Static cone penetration is internationally recognized as a standard field test to collect data about bearing capacity and frictional resistance of soil. The equipment meets essential requirements of IS: 4968 (Part III). The probing part consists of a cone which has an apex angle of 60^0 and overall base diameter of 35.7mm equivalent to is of $10cm^2$. The cone is connected with the sounding rods and mantle tubes. The assembly of sounding rod and mantle tubes is pushed into the soil, by means of hydraulic pumping unit and ram. Basically the test procedure consists of first pushing the cone and friction jacket and finally the whole assembly in sequence through specified depth and noting the resistance. The readings are taken at every 20cm of cone penetration and continuous data of end resistance and sleeve friction is recorded.

Static cone penetration test results: The procedure of conducting static cone penetration test is as per IS: 4968 (Part III) and ASTM D 5778. Total four number of static cone penetration test was conducted at 50m interval named as A, B, C, and D locations and continuous profile of cone resistance and sleeve friction is recorded. The graph shows the cone resistance and sleeve friction with respect to depth.

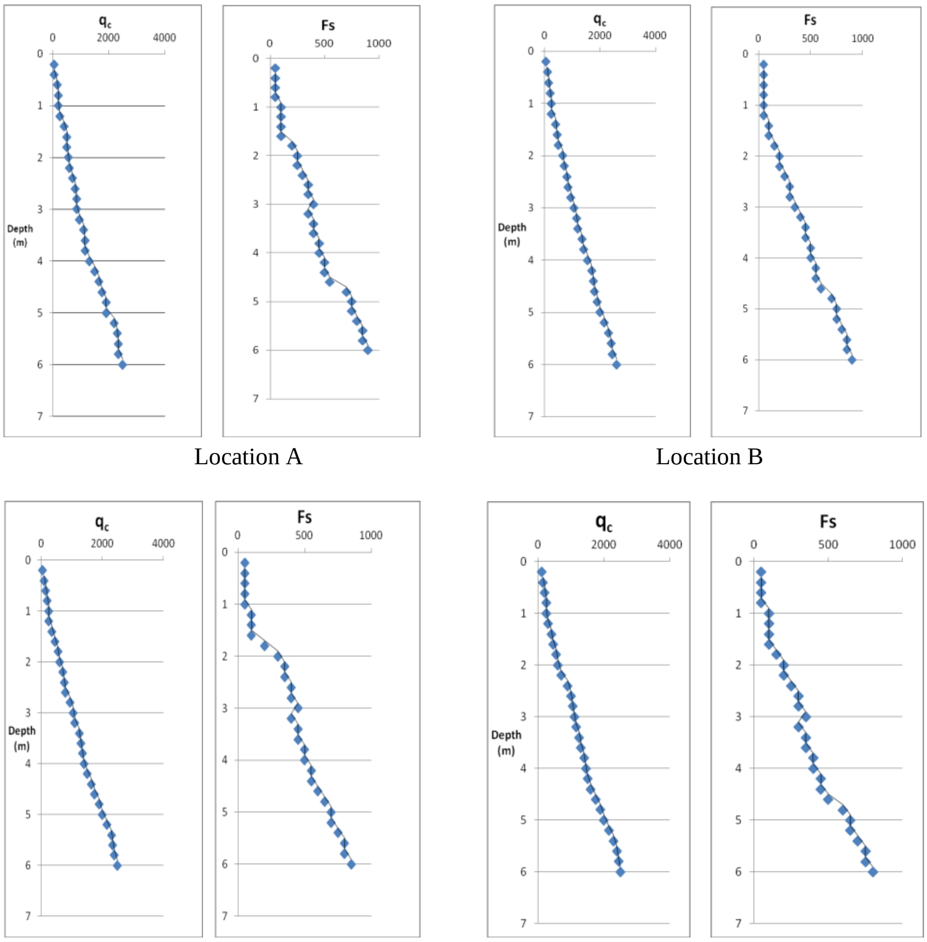


Fig. 2. Static cone penetration test results

From trends of the all graph it seems that cone resistance and frictional resistance goes on increasing with increase in depth. The range of cone resistance varies from 0 to 3000 kPa and range of frictional resistance varies from 0 to 1000 kPa. The resistances are recorded up to a depth of 6m for each 0.2m interval and are then converted into layers of 1.5m interval.

Table 1. Cone resistance and frictional resistance at every 1.5m interval

Location A			Location B		
Depth in m	q_c (kPa)	f_s (kPa)	Depth in m	q_c (kPa)	f_s (kPa)
0.00-1.50	225.00	75	0.00-1.50	281.25	62.5
1.50-3.00	692.85	300	1.50-3.00	785.71	250
3.00-4.50	1425.00	450	3.00-4.50	1487.50	500
4.50-6.00	2214.28	800	4.50-6.00	2257.14	800
Location C			Location D		
Depth in m	q_c (kPa)	f_s (kPa)	Depth in m	q_c (kPa)	f_s (kPa)
0.00-1.50	225.00	68.75	0.00-1.50	262.50	75
1.50-3.00	771.42	350	1.50-3.00	842.85	250
3.00-4.50	1412.50	500	3.00-4.50	1425.00	400
4.50-6.00	2228.57	750	4.50-6.00	2242.85	700

Corrections for static cone penetration test results: The combine cone and frictional resistance shall be corrected for the dead weight of the cone, frictional jacket and sounding rods. These values shall be corrected for the ratio of ram area to the base area of the cone.

I. Correction for cone resistance
   1. Mass of cone, m=0.8kg
   2. Mass of each sounding rod, m_1= 1.5kg
   3. Correction = ($m+nm_1$) x 10 kN/m^2

II. Correction for frictional resistance
   1. Mass of friction jacket m_f kg = 1.3kg
   2. Area of friction jacket, a= πdh = 113.04 cm^2
   3. Cone area at base, b= 10cm^2
   4. Correction factor to be added = $m_f/a + z$ ……. Where z= (f_s x b)/ a

III. Correction for pore pressure:

The recorded cone resistance and frictional resistance should be corrected for pore pressure as shown below,

$$q_t = q_c + u_2 (1-a)$$

a is the area ratio equals to 0.7 and u_2 is pore water pressure

Table 2. Corrected values of cone resistance and frictional resistance at every 1.5m interval

Location A			Location B		
Depth in m	q_c (kPa)	f_s (kPa)	Depth in m	q_c (kPa)	f_s (kPa)
0.00-1.50	256.27	7.73	0.00-1.50	261.91	6.63
1.50-3.00	748.62	27.64	1.50-3.00	840.9	23.22
3.00-4.50	1403.85	40.92	3.00-4.50	1572.01	45.34
4.50-6.00	2323.90	71.89	4.50-6.00	2366.15	71.89
Location C			Location D		
Depth in m	q_c (kPa)	f_s (kPa)	Depth in m	q_c (kPa)	f_s (kPa)
0.00-1.50	255.38	7.18	0.00-1.50	293.48	7.74
1.50-3.00	826.32	27.64	1.50-3.00	898.33	23.22
3.00-4.50	1496.72	45.34	3.00-4.50	1504.18	36.49
4.50-6.00	2343.85	67.47	4.50-6.00	2352.17	63

Sample Collection. The undisturbed and disturbed samples were collected at every 1.5m interval. The disturbed samples were collected using standard penetration test sampling tubes and undisturbed samples were collected through Shelby tubes of 100mm diameter and 0.9m length.

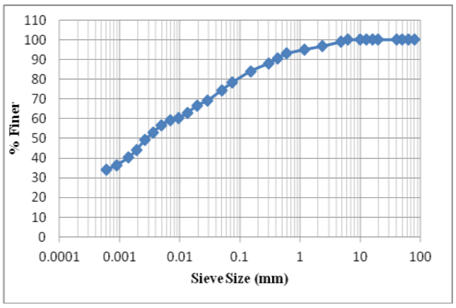
Fig.3. Collected disturbed and undistrurbed samples

4 Laboratory Testing

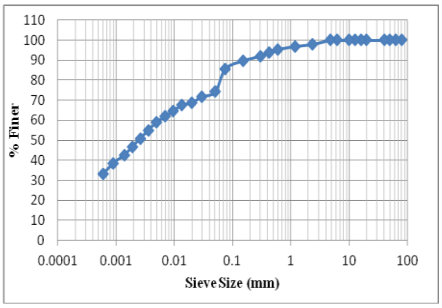
In order to determine index and engineering properties of the soil, various laboratory tests were conducted. The laboratory test program includes determination of specific gravity, grain size analysis, Atterberg's limits and undrained triaxial tests.

4.1 Grain size analysis

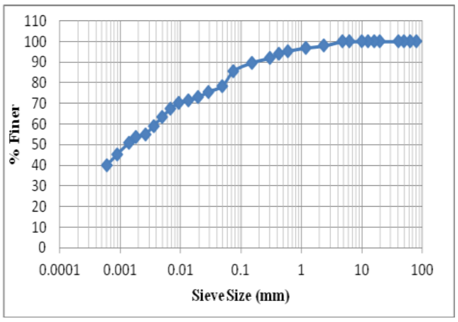
The grain size analysis and hydrometer test was conducted on collected soil samples of each location. Mostly for clay soil sample we t sieve analysis is preferred because it contains maximum amount of clay and silt particles. Hydrometer test was conducted on the soil samples which are less than 75μ. After conduction of tests the results of grain size analysis and hydrometer tests are club together in soil classification curve. Below graph shows the particle size distribution.



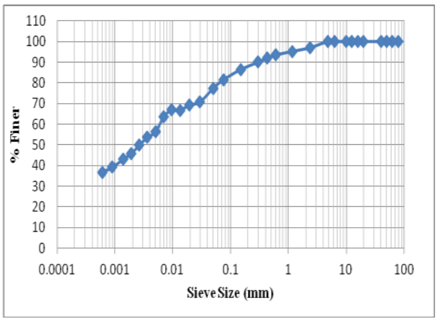
Location A



Location B



Location C



Location D

Fig. 4. Particle size distribution curve for location A, B, C & D

The particle size distribution results indicate the type of soil is silty clay. The content of clay in the soil is more than 80%.

4.2 Atterberg's limits

Atterberg's limits are basic tests to identify the behavior of the clay soil, which measures liquid limit, plastic limit and shriKage limit. As the water content in dry clayey soil increases it undergoes dramatic and distinct changes in behavior and consistency. Depending upon the presence of water content, it has four states like solid, semi-solid, plastic and liquid state. In each state behavior of soil and consistency is different and therefore engineering properties is also different in each state of clay soil. The results of consistency limits are tabulated below. The atterberg's limits are calculated at every 1.5m interval at each location.

Table 3. Results of Atterberg's Limit Test

Location A				
Depth in m	Liquid Limit	Plastic Limit	Shrinkage Limit	Plasticity Index
0.00-1.50	66	36	12.67	30
1.50-3.00	68	37.14	13.65	30.86
3.00-4.50	69.81	39.38	14.65	30.43
4.50-6.00	72	36	13.82	36

Location B				
Depth in m	Liquid Limit	Plastic Limit	Shrinkage Limit	Plasticity Index
0.00-1.50	67.5	33.82	12.67	33.68
1.50-3.00	69.25	36.02	13.65	33.23
3.00-4.50	70.25	37.67	14.65	32.58
4.50-6.00	71	35.65	13.82	35.35

Location C				
Depth in m	Liquid Limit	Plastic Limit	Shrinkage Limit	Plasticity Index
0.00-1.50	68	34.42	13.75	33.58
1.50-3.00	70	36.3	16.32	33.7
3.00-4.50	70.5	39	14.55	31.5
4.50-6.00	69	33.2	16.21	35.8

Location D				
Depth in m	Liquid Limit	Plastic Limit	Shrinkage Limit	Plasticity Index
0.00-1.50	67	33.8	14.3	33.2
1.50-3.00	69.5	31.92	14.9	37.58
3.00-4.50	68.5	36.3	15.25	32.2
4.50-6.00	70	33.97	16.2	36.03

4.3 Specific gravity

Specific gravity of the soil samples was determined using density bottle as per IS: 2720 (Part II) 1980. The tests were performed on collected soil samples from various layers. The values of specific gravity are shown in table below. The specific gravity of soil sample varies in between 1.58 to 1.64.

Table 4. Results of Specific Gravity Test

Depth in (m)	Specific Gravity Location A	Specific Gravity Location B	Specific Gravity Location C	Specific Gravity Location D
0.00-1.50	2.62	2.57	2.61	2.60
1.50-3.00	2.64	2.63	2.62	2.59
3.00-4.50	2.59	2.62	2.59	2.60
4.50-6.00	2.58	2.60	2.60	2.63

4.4 Triaxial test

The triaxial test on cylindrical sample is most widely used to determine undrained shear strength, bearing capacity parameters and stress strain relationship. In this research a strain controlled triaxial test are conducted as per procedure given in the IS: 2720 (Part 10, 11 & 12). Electrical load cell are used to measure the pore water pressure, displacement and volume change.

The triaxial tests were conducted on undisturbed soil samples collected from each location to determine undrained shear strength of soil. The strain rate was 0.002mm/min with different cell pressure of 0.5kg/cm^2, 1.0kg/cm^2 & 1.5kg/cm^2. The whole assembly of triaxial test is shown in figure below.

Fig.5. Collected undisturbed and disturbed samples

The results of triaxial test is tabulated below which shows that undrained shear strength of soil goes on increasing with increase in depth. The determined values of undrained shear strength of clay soil samples are then used for correlation with static cone penetration test data.

Table 5. Results of Specific Gravity Test

Location A		Location B	
Depth in (m)	Undrained Shear Strength (Su) kPa	Depth in (m)	Undrained Shear Strength (Su) kPa
0.00-2.00	18.49	0.00-2.00	16.67
1.60-3.20	52.34	1.60-3.20	54.96
2.60-4.60	99.43	2.60-4.60	104.94
4.20-6.50	140.47	4.20-6.50	147.08
Location C		Location D	
Depth in (m)	Undrained Shear Strength (Su) kPa	Depth in (m)	Undrained Shear Strength (Su) kPa
0.00-2.00	16.67	0.00-2.00	16.67
1.60-3.20	54.96	1.60-3.20	54.96
2.60-4.60	104.94	2.60-4.60	104.94
4.20-6.50	147.08	4.20-6.50	147.08

5 Determination of Cone Factor Values

It is very reliable estimation of undrained shear strength of clay from cone factor determined from the results of static cone penetration test. The value of cone factor is influenced by type of soil, test methods for determination of undrained shear strength. Many attempts have been made to fine cone factor values (Nash & Duffin, 1982; Lunne & Kelven, 1981; Chang et. al. 2001). From the studies it is observed that it is very necessary to calculate the cone factor values suitable for localized, specific type of soil considering all those factors. In this research we tried to calculate the cone factor values for the marine clay situated at Naigaon West, Mumbai. These cone factor values can be calculate using empirical relationships mentioned below.

5.1 Empirical cone factor values

There are many empirical correlations have been proposed to calculate the cone factor value. The researcher Terzaghi 1948 proposes a equation to calculate cone factor value based on recorded values of cone resistance and sleeve friction of static cone penetration test are

$$S_u = \frac{q_c - \sigma_{vo}}{K}$$

K = Empirical cone factor

σ'_{v0} = The total overburden stress.
The researcher Lunne in 1996 proposed a empirical relation to calculate cone factor from corrected tip resistance are as below,

$$S_u = \frac{q_t - \sigma_{vo}}{K_t}$$

Another equation has been proposed to calculate cone factor are as below,

$$S_u = \frac{q_t - u_2}{K_e}$$

From the empirical relationships mentioned above, the values of cone factor are calculated. These values of cone factor are ranging between 12 to 16 and an average of 14.7. The empirical cone factor values of every layer in each location are calculated and empirical relationship between index properties, engineering properties and cone factor has been made.

6 Correlation Between Cone Factor, Index Properties and Engineering Properties.

6.1 Correlation between Cone Factor and Plasticity Index

We are tried to plot the graph between all these three cone factor values K, Kt & Ke with plasticity index of the soil, we observed linear relationship between cone factors and plasticity index. Using correlation between cone factor and plasticity index of soil a localized equation Kt = 0.384Ip + 1.726 with a correlation coefficient equal to 0.926is suggested for estimating the cone factor. The analysis is meaningful as it indicated increasing trends of cone factor with plasticity index of clayey soil. The relationship between cone factor and plasticity index is shown in graph below.

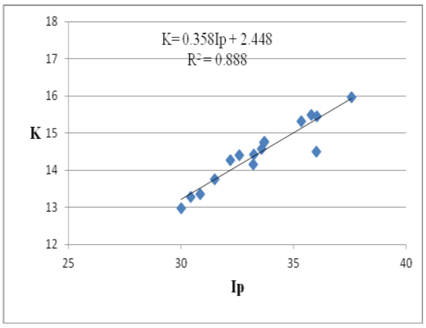


Fig.6. Correlation between K and plasticity index

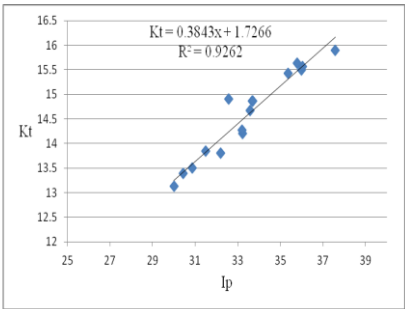


Fig.7. Correlation between Kt and plasticity index

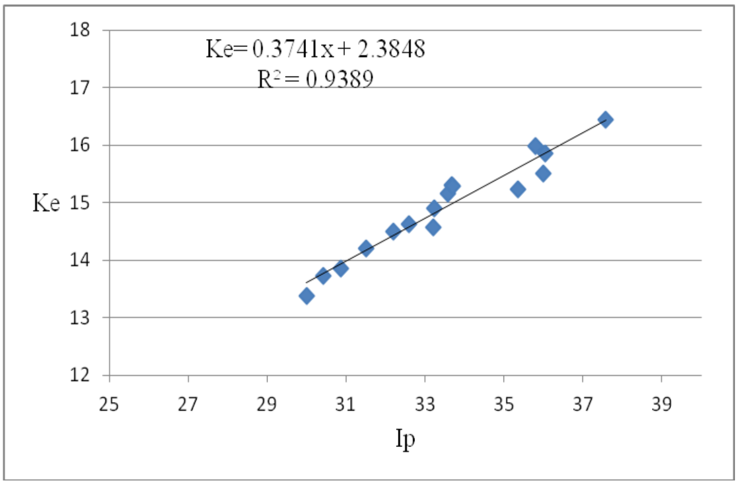


Fig.8. Correlation between Ke and plasticity index

6.2 Correlation between cone factor and undrained shear strength

To calculate in-situ undrained shear strength of soil using static cone penetration test, they must be correlate with each other. So, undrained shear strength of soil calculated using undrained triaxial test and cone resistance from cone penetration test on similar soil in Mumbai is plotted against each other as shown in graph. From this graph it seems that undrained shear strength of soil goes on increasing with increase in cone resistance of soil, so there is linear relationship between these two parameters.
Using the correlation between these two parameters a localized equation for undrained shear strength S_u= 0.064 (q_t - σ_{vo}) + 4.058 is suggested with correlation coefficient of 0.989, for the cone factor value of 14.87.

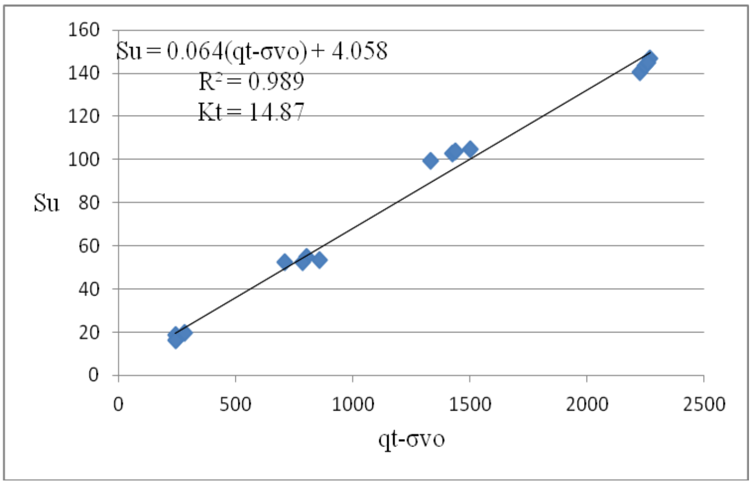


Fig.9. Correlation between Ke and plasticity index

7 Conclusions

1. From SCPT test results it is observed that Cone resistance and frictional resistance goes on increasing with increase in depth.
2. An empirical cone factor, K, K_t and K_e is derived from the empirical correlation and the values are in the range of 12 to 16 or average of 14.87.
3. The results of the present study showed increasing trends of cone factor with plasticity index of clayey soil.
4. Using the correlation between the cone factor and the plasticity index, a localized equation K =0.384Ip+1.726 with a correlation coefficient equal to 0.926 is suggested for estimating the cone factor.
5. Undrained shear strength can be estimated using static cone penetration test using formula Su = 0.064 ($q_t - \sigma_{vo}$) + 4.058 for cone factor value 14.87.

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