One-Point Plastic Limit Test via a Modified Fall-Cone Penetration Method

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Abstract

This paper developed a one-point testing method for determining the plastic limit of fine-grained soils using a fall cone penetration apparatus with a heavier, 240-gram cone, as proposed by C.P. Wroth and M.D. Wood (1978). The original method required multiple penetration trials at varying moisture contents to develop a regression line from points plotted on a semilogarithmic moisture content–penetration graph. The researcher simplified the testing procedure by deriving an equation that estimates the moisture content corresponding to a 20-millimeter penetration of the modified fall cone from a single instance of penetration. The value obtained, when combined with the liquid limit, can then be used to calculate the soil's plastic limit. Statistical analysis of multipoint modified fall cone plastic limit tests on was used. The one-point method developed yielded plastic limit values that are in close agreement with those obtained by the multipoint penetration method ($R = 0.8162$, $R^2 = 0.6662$, $T_{stat} < T_{crit}$)*. The values obtained via the one-point penetration method also had very little impact on the evaluation of the following geotechnical parameters: bearing capacity, consolidation, liquefaction, and AASHTO and USCS classifications. The method was also verified to be applicable to soils outside of the model equation's development set, yielding plastic limit values statistically equal to their multipoint counterparts* ($R = 0.8157, R^2 = 0.6654, T_{stat} < T_{crit}$)*. Lastly, statistically significant time and sample savings of 65.27% and 47.60%, respectively, were observed when using the one-point penetration method instead of the multipoint penetration method.*

Keywords*:* fall cone penetration, one-point, plastic limit, single point

1. Introduction

The Atterberg consistency limits, arbitrarily defined by Albert Mauritz Atterberg (1910, as cited in Tanyu *et al*., 2013) serve as one of the backbones of geotechnical and foundation engineering. The limits serve as the boundaries upon which the soil transitions from one engineering state to the other. One of the consistency limits defined by Atterberg (1910, as cited in Das & Sobhan, 2018) is the plastic limit, which is the minimum water content required for the soil to start exhibiting plastic behavior. Although non-destructive methods for soil index properties determination have been put forward such as Electrical Resistivity Tomography (ERT) (Mohammed *et al*., 2019; Sangprasat *et al*., 2024), the correlations presented are usually site-specific and lack are not recommended by any standards committee or agency.

Currently, only one testing method for the determination of soils' plastic limit has been standardized by both the American Society for Testing and Materials (ASTM) and the British Standards Institute (BSI), the soil thread rolling method. However, it has one major problem: it is highly susceptible to operator variability and exhibits low repeatability (Vardanega *et al*., 2022). An alternative to this method is the modified fall cone penetration method proposed by Wroth & Wood (1978), a modification of the fall cone penetration method proposed by Hansbo (1957) for determining soils' liquid limits. Wroth and Wood's (1978) method was successful at reducing the variability of the thread rolling method (O'Kelly *et al*., 2020) but has inherited two major problems inherent in the multi-point fall cone penetration test for liquid limit. First, the amount of soil sample required is relatively large when compared to the amount required for the soil thread rolling method (Sridharan & Nagaraj, 2004). Second, the minimum testing time recommended cannot fall below 40 minutes (British Standards Institution, 1990).

To remedy the problems inherent in the multi-point, fall cone penetration test for liquid limit determination proposed by Hansbo (1957, as cited by Clayton & Jukes, 1981), a one-point test was proposed from and analysis of 582 soil samples. Their method has now been adopted by the latest edition of the BSI for soil classification as an alternative to the multi-point cone penetration test for liquid limit determination (British Standards Institution, 1990). Additionally, an alternative method for liquid limit determination, the multi-point percussion cup method, developed by Arthur Casagrande (1932, as cited in Das & Sobhan, 2018) also has its corresponding one-point method. The method was developed by the U.S. Army Corps of Engineers, Waterways

Experiment Station in 1949 from their analysis of 732 samples (U.S. Army Corps of Engineers, Waterways Experiment Station, 1949). Today, the one-point percussion cup method developed by the U.S. Army Corps of Engineers, Waterways Experiment Station has also been approved and standardized by the ASTM (ASTM, 2017).

Since there are one-point liquid limit methods for both the Casagrande percussion cup and the fall cone penetration method, the researcher believes that there should also exist at least a possibility of developing a one-point method for the determination of soils' plastic limits. Additionally, the researcher deemed the modified fall cone method proposed by Wroth and Wood (1978) as the most viable alternative to the current thread rolling method, and hence being the basis for deriving a one-point method for plastic limit determination, for three reasons: the method was derived with respect to soil shear strength; the plastic limit values obtained had minimal deviation from those obtained via the percussion cup method; and there is a wide global adoption of the British standard cone geometry in fall cone apparatus.

First, in their derivation, the researchers assume that there is a hundred-fold increase in shear strength between the soil's liquid limit state and plastic limit state (Wroth & Wood, 1978). This assumption was originally obtained by the previous researchers from an even earlier study on fine-grained clays. The proponents of this earlier paper found out that at the soil's plastic limit, the soil has approximately a hundred times more shear strength than when it is at its liquid limit (Skempton & Northey, 1952). This was seconded by a more recent study still focused on clays. The updated study concludes that for correlations involving the variation of strength and stiffness of the soil with respect to its water content, it would be favorable for the plastic limit to be defined as the water content at which there is a hundred-fold increase in shear strength with respect to the liquid limit (Haigh *et al.*, 2014).

Second, the similarity between the results of plastic limit tests using the modified fall cone and the thread rolling method was also highlighted in their paper, with the average percentage error between the two methods being only 4% (Wroth & Wood, 1978).

Lastly, the modified fall cone proposed by Wroth and Wood (1978) had similar geometry with the British standard 30 – degree fall cone which is widely adopted globally by soil testing laboratories. This wide adoption is highlighted by a recent paper stating that an internationally standardized fall cone setup should consider using the 30 – degree cone as it is the most widely used fall cone geometry (O'Kelly *et al*., 2020). This argument is further supported by a more recent study that aimed to predict the undrained shear strength of a soil using a standard 30-degree British fall cone apparatus (Dastider *et al*., 2021).

2. Methodology

2.1 Scope and Limitations of the Samples and Testing Methodologies

Like previous studies, which developed one-point testing methods, laboratory test data of soils tested via the corresponding multi-point method were used. In this case, the test data of soils which were tested via the multi-point modified fall cone penetration method were used. Furthermore, the soils that were to be included in the study had to meet the following criteria:

- At least 50% of soil particles in the sample shall pass US standard sieve No. 200. This is because the method upon which this study is based has been conducted exclusively on fine-grained soils (Wroth & Wood, 1978).
- The Atterberg consistency test results fall on or below the USCS "U"-Line because soil samples not conforming to this most likely have erroneous test results (ASTM, 2017)

All the soil samples included in this study were gathered from regions 10, 11, and 12 of Mindanao, Philippines. Furthermore, all laboratory tests conducted on the soil samples were performed at AC Joyo Design and Technical Services, a Department of Public Works and Highways – Bureau of Research and Standards (DWPH-BRS) accredited engineering consultancy firm that specializes in geotechnical engineering and subsurface exploration; of which the researcher is a part of. Lastly, the confidentiality of the clients from which the soil sample results have been obtained is given the utmost priority and any information contained by the samples that may prove detrimental to the client if misused are omitted or censored. This includes, but is not limited to, the client's name, the site's exact geographical location, the proposed structure's design parameters, and the project name.

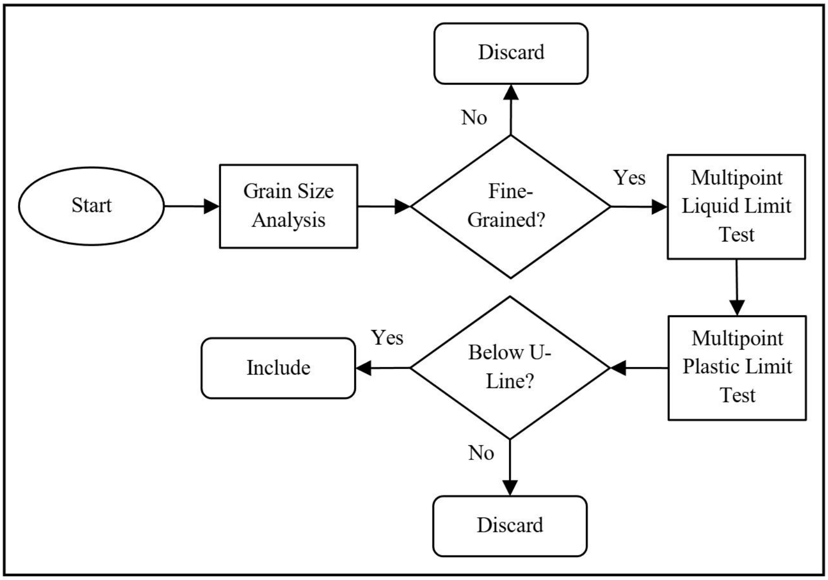

Figure 1. Flowchart for classification of soils to be included in the study

2.2 Discussion on the Multipoint Penetration Method

The modified fall cone penetration method proposed by Wroth and Wood (1978) calculates the plastic limit (PL) of a soil sample with Equation 1:

$$PL = LL - \frac{2\Delta}{log\,3} \qquad (1)$$

where LL stands for the liquid limit, obtained via the multi-point fall cone penetration test for liquid limit and Δ represents the constant vertical distance between the semilogarithmic best-fit lines of the liquid and plastic limit tests, as shown in Figure 2.

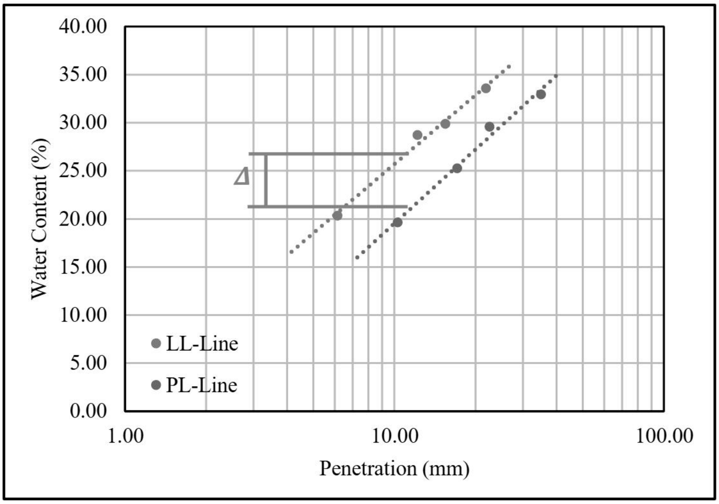

Figure 2. Example semilogarithmic plot of liquid and plastic limit test

Since the liquid limit was defined by Hansbo (1957) as the water content at which the British standard 30-degree cone would penetrate 20 mm into the soil, it would be easier to define Δ as the difference between the liquid limit and the water content corresponding to a penetration of 20 mm when using the modified fall cone for plastic limit determination (PLL_{20}):

$$\Delta = LL - PLL_{20} \tag{2}$$

Substituting Equation 2 into Equation 1 yields the following Equation 3:

$$PL = LL - \frac{2(LL - PLL_{20})}{\log 3} \tag{3}$$

The semilogarithmic best-fit lines for both the liquid and plastic limit tests shown in Figure 2 can be expressed with Equation 4:

$$w = A \log P + B \tag{4}$$

where P stands for the penetration of the cone into the soil, in mm, w represents the water content corresponding to the observed penetration, in percent, and A and B are the gradient and y-intercept of the regression line, respectively. Therefore, PLL_{20} can also be described by Equation 5:

$$PLL_{20} = A_{PL} \log 20 + B_{PL} \tag{5}$$

where A_{PL} and B_{PL} are the gradient and y-intercept, respectively, of the regression line generated specifically from the multi-point modified fall cone penetration test for plastic limit determination.

2.3 Derivation of the Study's Assumptions and Sample Size

The sample size, together with two working assumptions for this research were obtained by statistical analysis of a preliminary data set, as suggested by Miot (2011). The preliminary data set, shown in Table 1, consisted of 30 randomly selected soil samples that were tested via the multi-point modified fall cone penetration test. From statistical analysis of the preliminary data, two working assumptions were generated.

The coefficient of variation (COV) of A_{PL} was calculated to be equal to 24.99%. This denotes that the variations in the values of A_{PL} are acceptably close to each other end tend to converge towards a single value close to the mean (Selvamuthu & Das, 2018). Hence,

1. The A_{PL} for all fine-grained soils were approximately equal.

Additionally, the coefficient of variation of B_{PL} was calculated to be equal to 339.64%. Therefore, there does not appear to exist a central value at which subsequent measurements of B_{PL} would converge (Selvamuthu & Das, 2018). Therefore,

2. The B_{PL} for different fine-grained soil samples were significantly different from one another. However, the B_{PL} of the same sample remains constant throughout the duration of each of their respective multipoint plastic limit tests.

Table 1. Preliminary multi-point modified fall cone penetration data set

No	Site	Sample	LL	A_{PL}	B_{PL}	PL_4
1	SITE 1	BH1-S4	35.93	26.85	-4.81	11.53
2	SITE 2	BH2-S1	61.22	43.63	-4.26	24.67
3	SITE 3	BH5-S1	59.23	44.86	-7.30	24.99
4	SITE 4	TP1-2	61.06	44.82	-7.28	19.01
5	SITE 4	TP2-2	84.83	53.87	1.61	29.80
6	SITE 4	TP3-1	64.56	46.83	-6.12	23.67
7	SITE 4	TP4-2	44.00	24.00	7.53	22.02
8	SITE 4	BH1-S1	47.04	31.48	-0.87	17.91
9	SITE 4	BH12-S1	55.12	37.67	-2.50	19.06
10	SITE 4	BH12-S3	53.21	33.20	2.46	21.52
11	SITE 4	BH14-S3	37.63	23.29	2.31	16.59
12	SITE 4	BH14-S4	68.84	50.93	-6.17	32.14
13	SITE 4	BH15-S1	54.36	35.04	1.74	24.89
14	SITE 4	BH15-S2	55.95	38.33	-1.40	24.57
15	SITE 4	BH16-S2	54.12	36.22	-2.29	15.19
16	SITE 5	TPC1-1	55.44	31.50	9.42	34.32
17	SITE 5	TPC2-1	59.87	28.82	16.82	36.60
18	SITE 5	TPC3-2	56.72	50.71	-17.55	21.91
19	SITE 5	TPC4-1	72.73	29.99	26.18	41.15
20	SITE 5	TPC4-2	70.20	23.81	33.49	46.16
21	SITE 5	TPC5-2	75.44	39.36	16.78	44.20
22	SITE 5	TPC6-1	73.15	52.05	-3.39	36.18
23	SITE 5	TPSG1-2	51.40	29.40	5.75	20.39
24	SITE 5	TPSG2-1	65.01	50.66	-8.40	33.58
25	SITE 5	TPSG2-2	46.89	33.03	-2.22	21.15
26	SITE 5	TPSG5-1	90.13	49.19	15.28	44.64
27	SITE 5	TPSG5-2	83.00	31.29	34.29	49.45
28	SITE 5	H1-S1	55.22	31.67	6.88	25.31
29	SITE 5	H1-S2	52.45	34.37	0.95	24.00
30	SITE 5	SP1-S1	74.94	49.28	0.31	30.86
Mean:			60.66	37.87	3.57	27.92
Standard Deviation:			13.24	9.47	12.14	9.95
COV (%):			21.82	24.99	339.64	35.66

Building upon the first assumption previously established, the theoretical central value that all tested soils' A_{PL}'s converge into may be denoted by A_{ave}, which is just equal to the mean of the semilogarithmic regression lines' gradients (Equation 6):

$$A_{ave} = \frac{\sum A_{PL}}{n} \tag{6}$$

Therefore, the general equation of the semilogarithmic regression line for all multi-point fall cone penetration tests can be written as Equation 7, a modified form of Equation 4:

$$w = A_{ave} \log P + B_{PL} \tag{7}$$

or more conveniently,

$$B_{PL} = w - A_{ave} \log P \tag{8}$$

Furthermore, Equation 5, which describes the water content at which a penetration of 20 mm is achieved by the modified fall cone can also be re-written as Equation 9:

$$PLL_{20} = A_{ave} \log 20 + B_{PL} \tag{9}$$

or more conveniently,

$$B_{PL} = PLL_{20} - A_{ave} \log 20 \tag{10}$$

The second established assumption can then be used to combine Equation 8 and Equation 10 which simplifies into:

$$PLL_{20} = w + A_{ave} \left(\log \frac{20}{P}\right) \tag{11}$$

Once a reliable value of A_{ave} has been obtained, the derived equation, Equation 11 can successfully estimate the moisture content corresponding to a penetration of 20 mm (PLL_{20}) given only a single measurement of penetration (P) and the corresponding its water content (w). Combining this equation with equation Equation 3, which calculates for the plastic limit, a model equation that can obtain the estimated plastic limit (PL_1) with only a single instance of penetration can then be obtained:

$$PL_1 = LL - \frac{2\left(LL - \left(w + A_{ave}\left(\log \frac{20}{P}\right)\right)\right)}{\log 3} \tag{12}$$

To finalize the model equation (Equation 12), it is integral to obtain a reliable value for A_{ave}. To achieve this, an appropriate sample size is required. For an infinite population, with available preliminary sample data, Miot (2011) gives Equation 13 to obtain the sample size:

$$n = \left(\frac{Z\delta}{E}\right)^2 \tag{13}$$

where Z is the critical value for the desired degree of confidence, (δ) is the standard deviation of the preliminary data, and E is the margin of error, usually a percentage of the preliminary data's mean. Therefore, for a desired confidence level of 95% ($Z = 1.96$), and a desired margin of error of 7% of the preliminary data's mean ($E = 2.65$), together with the calculated standard deviation of A_{PL} which is equal to $\delta = 9.47$, the sample size can be calculated as:

$$n = \left(\frac{1.96 \times 9.47}{2.65}\right)^2$$

$$n = 49.03 \approx 50$$

2.4 Validation of the Model Equation's Estimated Plastic Limit

After obtaining a reliable value of A_{ave} and therefore finalizing the model equation (Equation 12), statistical tests must be performed to evaluate the strength of the equation in predicting the plastic limit. For this, the mean absolute error (MAE), Pearson's correlation coefficient (R), Pearson's coefficient of determination (R^2), and two-tailed paired statistical T-test with a level of significance ($\alpha = 0.05$) were utilized. The plastic limit values obtained via multi-point modified fall cone penetration testing (PL_4) were compared with those obtained via one-point modified fall cone penetration testing (PL_1). The four statistical tests were performed on both the complete data set used to develop the model equation, which consists of 50 samples, and an independent validation data set, which consists of 10 samples.

Since the laboratory data readily available all made use of the multi-point method, each sample utilizing 4 penetration trials at 4 distinct water contents, only the penetration closest to 20 mm (P') and the water content at which it was observed (w') were used in obtaining the estimated plastic limit (PL_1) via the generated final form of Equation 12. This reasoning was based upon the one-point liquid limit test methodology proposed by previous researchers (Clayton & Jukes, 1981) which suggested relying on operator experience to obtain soil consistency which was enough to yield a desired penetration or blow count close to their respective target values during testing.

2.5 Assessing the Impact of One-point Fall Cone Penetration on Geotechnical Parameters

Additionally, the impact of utilizing the estimated plastic limit obtained via the one-point modified fall cone penetration method on the calculation or evaluation of the following geotechnical parameters was also evaluated:

- Bearing capacity
- Consolidation settlement
- Liquefaction susceptibility
- American Association of Highway and Transportation Officials (AASHTO) Soil Classification System
- Unified Soil Classification System (USCS)

For the bearing capacity and consolidation settlement analyses, idealized shallow footing conditions were assumed. The footing dimensions, loading conditions, soil properties and soil stratification were all obtained from previous studies which presented ranges for the parameters. The studies all aimed to assess either the bearing capacity or consolidation settlement from in-situ data. This measure was done to ensure that the footing conditions presented in this research for the purpose of validating the one-point modified fall cone penetration method for plastic limit determination, although idealized, were still in-line with expected in-situ conditions. Since bearing capacity and consolidation settlement are quantitative variables, the four statistical tests used to quantify the predictive strength of the model were also used, specifically, the mean absolute error (MAE), Pearson's correlation coefficient (R), Pearson's coefficient of determination (R^2), and two-tailed paired statistical T-test with a level of significance ($\alpha = 0.05$).

For the liquefaction susceptibility, AASHTO soil classification, and USCS analyses there was no need to generate idealized foundation or loading conditions. Since the parameters were qualitative, only the accuracy of the evaluations was measured. That is, the evaluations made from utilizing the one-point plastic limit values were compared to the evaluations made when using the multi-point plastic limit values.

2.6 Assessing the Impact of One-point Fall Cone Penetration on Testing Time and Sample Amount

Finally, the variation in testing time and sample amount required to perform both the multi-point and one-point modified fall cone penetration tests was also measured. For this, 10 additional soil samples were tested via both multi-point and one-point penetration methods. The tests were timed, and the amount of sample used were also weighed. To assess the impact of utilizing the one-point modified fall cone method for these test variables, two-tailed paired statistical T-testing with a level of significance ($\alpha = 0.05$) was also used to determine the significance of the difference between the mean testing time and mean sample amount for the two methods in scrutiny.

3. Results and Discussion

3.1 Derivation and of the Final Model Equation

As derived earlier, the required sample size to obtain the desired value of (A_{ave}) is 50. Since the preliminary data already contained 30 samples, only an additional 20 samples tested via the modified fall cone penetration test were required to complete the sample size. These are presented in Table 2.

Table 2. Additional multi-point modified fall cone penetration data set

No	Site	Sample	LL	A_{PL}	B_{PL}	PL_4
31	SITE 6	BH1-S3	69.17	39.13	6.41	19.50
32	SITE 7	BH2-S5	45.61	27.83	2.90	18.37
33	SITE 8	BH1-S5	55.21	22.57	19.83	29.97
34	SITE 8	BH2-S3	66.08	38.51	6.05	24.48
35	SITE 9	BH1-S3	54.47	22.09	19.54	28.54
36	SITE 10	BH1-S1	63.00	27.38	21.26	37.39
37	SITE 11	BH1-S2	55.68	36.85	3.57	38.23
38	SITE 12	BH1-S3	61.51	38.60	1.78	21.67
39	SITE 12	BH1-S6	47.50	35.87	-5.80	19.72
40	SITE 12	BH2-S3	34.05	22.64	0.24	15.79
41	SITE 13	BH2.1-S8	41.54	26.14	2.59	20.83
42	SITE 14	Rice	46.24	26.26	7.27	26.10
43	SITE 15	Banana	53.02	26.97	11.91	27.78
44	SITE 16	Rice	37.08	29.93	-6.45	17.82
45	SITE 17	Banana	41.95	24.84	3.67	16.96
46	SITE 18	Rubber	52.91	28.55	8.54	22.58
47	SITE 19	Banana	53.20	27.55	11.55	28.89
48	SITE 20	Coco	47.23	22.45	12.49	24.04
49	SITE 21	Rice	38.51	23.61	3.09	18.79
50	SITE 22	Corn	45.99	30.12	0.28	18.66

Taking the average value of the A_{PL}'s for both the preliminary and the additional data sets, a value of A_{ave} can be calculated by Equation 6. Additionally, the calculated value of A_{ave} can then be substituted into Equation 10. Simplifying, the final model equation for the one-point modified fall cone penetration test for plastic limit determination can then be obtained:

$$PL_1 = -3.19LL + 4.19w' + 143.63 \log\left(\frac{20}{P'}\right) \tag{14}$$

3.2 Validation of the Final Model Equation

To validate the strength of the model equation, it was necessary to calculate the estimated plastic limit (PL_1) of the complete data set. As discussed, the penetration closest to 20 mm (P') and the water content at which it was observed (w') were used for this. Table 3 tabulates the necessary values, together with the multi-point (PL_4) and one-point (PL_1) plastic limit values. The error of the one-point plastic limit (PL_1) as a percentage of the multi-point plastic limit are also shown together with the mean absolute error (MAE). For conciseness, the identifier columns "Site" and "Sample" were omitted in the following tabulation.

Table 3. Model equation generation set validation

No	PL_4	w'	P'	PL_1	$\%Error$
1	11.53	33.58	21.16	22.56	95.60
2	24.67	50.74	18.43	22.38	-9.28
3	24.99	52.52	22.09	24.89	-0.40
4	19.01	46.35	18.17	5.40	-71.58
5	29.80	74.00	22.89	31.01	4.05
6	23.67	56.03	20.66	26.80	13.23
7	22.02	39.87	22.17	20.28	-7.89
8	17.91	39.82	20.32	15.79	-11.82
9	19.06	44.62	18.91	14.57	-23.53
10	21.52	42.18	17.11	16.70	-22.42
11	16.59	32.08	18.09	20.61	24.27
12	32.14	59.09	19.80	28.61	-11.00
13	24.89	44.97	18.45	20.02	-19.58
14	24.57	48.89	20.34	25.30	2.94
15	15.19	40.83	17.28	7.53	-50.44
16	34.32	47.16	16.59	32.40	-5.61
17	36.60	53.38	21.01	29.60	-19.14
18	21.91	48.82	20.23	22.88	4.42
19	41.15	69.64	23.26	50.38	22.43
20	46.16	62.76	21.01	35.92	-22.18
21	44.20	70.59	22.96	46.49	5.17
22	36.18	63.35	19.08	35.02	-3.22

Table Continued.

23	20.39	44.44	19.88	22.61	10.86
24	33.58	54.29	18.05	26.47	-21.18
25	21.15	39.38	18.96	18.72	-11.49
26	44.64	76.64	17.98	40.23	-9.88
27	49.45	77.46	27.52	39.87	-19.37
28	25.31	48.47	21.96	21.08	-16.69
29	24.00	44.17	18.59	22.31	-7.02
30	30.86	60.87	17.45	24.47	-20.69
31	19.50	55.09	17.45	18.66	-4.30
32	18.37	39.88	16.73	32.75	78.28
33	29.97	50.00	23.74	22.67	-24.36
34	24.48	54.50	20.54	15.90	-35.06
35	28.54	52.03	26.35	27.03	-5.29
36	37.39	57.23	22.05	32.72	-12.49
37	38.23	52.80	19.87	44.03	15.16
38	21.67	50.88	19.00	20.15	-6.98
39	19.72	42.92	19.66	29.37	48.93
40	15.79	30.52	24.30	7.10	-55.02
41	20.83	35.80	19.08	20.39	-2.10
42	26.10	41.72	20.76	24.96	-4.38
43	27.78	45.71	19.78	23.07	-16.97
44	17.82	34.21	21.51	20.49	15.02
45	16.96	35.11	20.57	11.55	-31.91
46	22.58	44.44	19.16	20.10	-10.98
47	28.89	48.03	18.92	34.99	21.11
48	24.04	40.24	18.36	23.28	-3.15
49	18.79	34.33	22.90	12.57	-33.08
50	18.66	42.21	23.02	21.38	14.61
		MAE (%)			20.13

Additionally, a graph showing the multi-point plastic limit (PL_4) against the one-point plastic limit (PL_1) is shown in Figure 3.

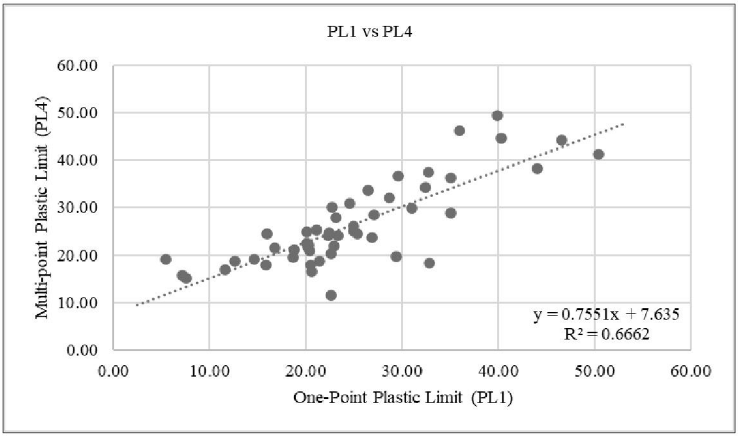

Figure 3. Regression plot of the multi-point plastic limit (PL_4) as a function of the one-point plastic limit (PL_1)

It can be seen in Figure 3 that the coefficient of determination (R^2) between the plastic limit values obtained via the two methods was 0.6662. The result denotes that a large percentage (66.62%) of the total variation in the data can be explained by the proposed model and that the proposed model (Singh, 2006). Furthermore, the correlation coefficient (R) can be calculated as the square-root of (R^2), which is equal to 0.8162. This implies a strong positive correlation between the two sets of data (Singh, 2006). It was also determined that the one-point plastic limit (PL_1) was within +2.91% to -13.09% of the multi-point plastic limit (PL_4) at a confidence level of 95%.

Lastly, a two-tailed paired T-test was performed to compare the means of the two data sets. Since the calculated t-value (1.9949) is less than the critical t-value (2.0096), it can be said that there does not exist a significant statistical difference in the means of the plastic limits obtained by the two methods.

To assess the viability of the model equation to samples not included in the generation data set, a validation data set comprising of 10 independent samples, shown in Table 4, was used.

Table 4. Validation data set

No	Site	Sample	PL_4	w'	P'	PL_1
1	SITE 4	TP5-2	29.62	45.65	21.17	28.44
2	SITE 6	BH2-S4	37.62	62.66	16.29	31.31
3	SITE 7	BH1-S8	29.19	36.95	20.7	27.72
4	SITE 14	Rubber	20.97	31.11	21.1	18.49
5	SITE 20	Rice	21	35.71	18.1	25.79
6	SITE 15	Coco	23.55	35.82	19.46	24.11
7	SITE 29	Rubber	19.44	31.89	20.37	21.94
8	SITE 31	Rice	22.28	41.79	23.01	22.33
9	SITE 32	Corn	21.25	38.24	25.1	21.31
10	SITE 33	Rice	21.55	31.55	20.86	16.31

The statistical analysis tools used in the back-validation of the generation data set were also used for the validation data set. The mean absolute error (MAE) of the sample can be computed to be equal to 10.04%. A regression graph showing (PL_4) as a function of (PL_1) is also shown in Figure 4. The coefficient of determination (R^2) was determined to be 0.6654 whilst the correlation coefficient (R) can be calculated as 0.8157. For the validation data set, it can be calculated that (PL_1) was within -12.14% to +7.47% of (PL_4) at a confidence level of 95%.

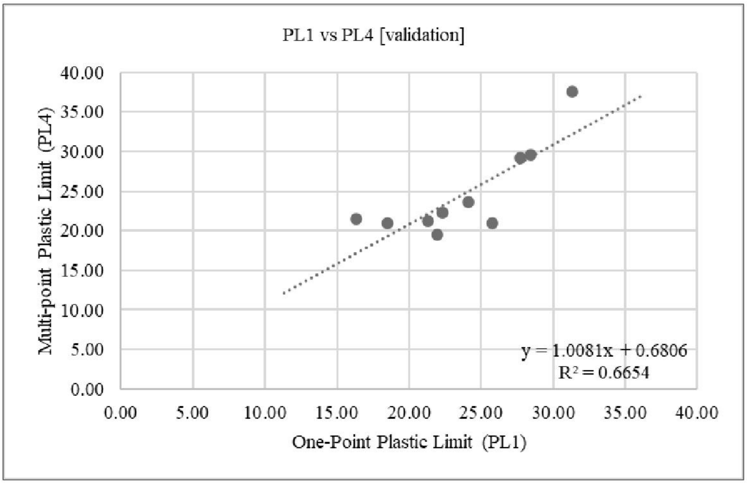

Figure 4. Regression plot of the multi-point plastic limit (PL_4) as a function of the one-point plastic limit (PL_1) for the validation set

Two-tailed paired T-testing was also performed for the validation set. From the results of the t-Test, it is evident that the calculated t-value (0.8334) is less than the critical T-value (2.2622). Therefore, it can be concluded that there does not exist a statistically significant difference between the plastic limits obtained via the one-point penetration and the multi-point penetration tests and that the values obtained via the two methods are close to each other.

3.2 Assessment of Impact on Geotechnical Parameters

Table 5. Bearing capacity and settlement impact assessment

No	Bearing Capacity					Settlement				
	PL_4		PL_1		Error	PL_4		PL_1		Error
	UBC_4	ABC_4	UBC_1	ABC_1	(%)	Cc_4	Sc_4	Cc_1	Sc_1	(%)
1	1,725	575	2,547	849	2.03	0.028	35.10	0.016	20.46	-41.71
2	1,212	404	1,119	373	3.25	0.038	48.39	0.041	51.43	6.28
3	1,311	437	1,305	435	3.01	0.036	45.55	0.036	45.68	0.29
4	1,014	338	645	215	4.72	0.044	55.70	0.058	73.77	32.44
5	657	219	675	225	2.92	0.055	70.26	0.054	68.65	-2.28
6	1,047	349	1,161	387	2.71	0.042	53.78	0.039	49.62	-7.73
7	1,836	612	1,743	581	3.16	0.024	30.98	0.026	33.29	7.45
8	1,545	515	1,455	485	3.19	0.032	40.14	0.034	42.95	7.00
9	1,233	411	1,065	355	3.47	0.038	48.43	0.043	54.38	12.29
10	1,431	477	1,212	404	3.54	0.034	42.83	0.039	49.24	14.96
11	1,896	632	2,181	727	2.61	0.024	30.45	0.020	25.11	-17.55
12	1,209	403	1,071	357	3.39	0.038	47.72	0.041	52.41	9.83

Table Continued.

13	1,530	510	1,305	435	3.52	0.031	39.75	0.036	46.23	16.28
14	1,440	480	1,485	495	2.91	0.033	42.11	0.032	41.15	-2.27
15	1,116	372	864	288	3.88	0.041	52.34	0.049	62.51	19.44
16	1,896	632	1,782	594	3.19	0.023	28.55	0.025	31.10	8.95
17	1,782	594	1,506	502	3.55	0.024	30.90	0.032	40.20	30.10
18	1,284	428	1,329	443	2.90	0.037	46.58	0.036	45.30	-2.76
19	1,437	479	1,818	606	2.37	0.032	40.49	0.022	28.24	-30.26
20	1,743	581	1,305	435	4.01	0.024	30.78	0.035	44.37	44.16
21	1,458	486	1,545	515	2.83	0.031	39.74	0.029	36.70	-7.64
22	1,200	400	1,155	385	3.12	0.038	47.60	0.039	49.15	3.25
23	1,461	487	1,560	520	2.81	0.033	42.14	0.031	39.20	-6.98
24	1,437	479	1,137	379	3.79	0.032	41.16	0.040	50.60	22.94
25	1,674	558	1,575	525	3.19	0.028	35.64	0.031	38.87	9.05
26	900	300	774	258	3.49	0.045	57.01	0.050	62.86	10.27
27	1,338	446	972	324	4.13	0.033	41.95	0.043	54.67	30.32
28	1,515	505	1,311	437	3.47	0.032	40.25	0.036	45.86	13.93
29	1,560	520	1,509	503	3.10	0.030	38.63	0.032	40.87	5.79
30	945	315	741	247	3.83	0.045	56.85	0.052	65.32	14.91
31	783	261	741	247	3.17	0.051	64.92	0.052	66.03	1.71
32	1,608	536	2,634	878	1.83	0.030	37.78	0.015	18.69	-50.54
33	1,692	564	1,386	462	3.66	0.027	34.05	0.035	43.74	28.47
34	1,020	340	747	249	4.10	0.043	54.55	0.052	65.94	20.89
35	1,656	552	1,608	536	3.09	0.028	35.05	0.029	37.05	5.71
36	1,674	558	1,506	502	3.33	0.027	33.66	0.031	39.86	18.43
37	2,133	711	2,823	941	2.27	0.019	23.65	0.013	15.96	-32.53
38	1,086	362	1,038	346	3.14	0.042	52.72	0.043	54.73	3.81
39	1,590	530	2,088	696	2.28	0.030	38.28	0.020	25.47	-33.47
40	2,064	688	1,623	541	3.82	0.021	27.16	0.031	38.70	42.47
41	1,917	639	1,875	625	3.07	0.023	29.58	0.024	30.16	1.97
42	1,935	645	1,875	625	3.10	0.022	28.29	0.024	29.81	5.36
43	1,692	564	1,515	505	3.35	0.027	34.29	0.032	40.55	18.25
44	1,998	666	2,202	734	2.72	0.022	28.16	0.019	24.61	-12.62
45	1,692	564	1,491	497	3.40	0.028	35.21	0.033	42.39	20.41
46	1,506	502	1,377	459	3.28	0.032	41.07	0.035	44.36	8.01
47	1,725	575	2,088	696	2.48	0.026	33.03	0.020	24.94	-24.51
48	1,782	594	1,743	581	3.07	0.025	32.22	0.026	33.23	3.12
49	1,977	659	1,656	552	3.58	0.023	28.59	0.029	36.85	28.86
50	1,608	536	1,710	570	2.82	0.030	37.86	0.027	34.24	-9.56

3.2.1 Allowable Bearing Capacity

To ascertain whether the results of the proposed single point modified fall cone penetration plastic limit method's results yield sufficiently accurate enough values for use in calculation of soils' bearing capacities, the researcher generated an idealized scenario not dissimilar to actual site cases that require a geotechnical engineer to analyse the long-term (consolidated – drained) bearing capacity of a shallow foundation in over-consolidated clay. A sketch of the foundation dimensions, soil profile, and other assumptions are shown in Figure 5. The shallow foundation geometry assumed by the researcher falls within the range of measurements obtained by Shahin *et al*. (2002) and Tezcan *et al*. (2006) who have studied the in-situ behaviour or properties of actual shallow foundations in their studies.

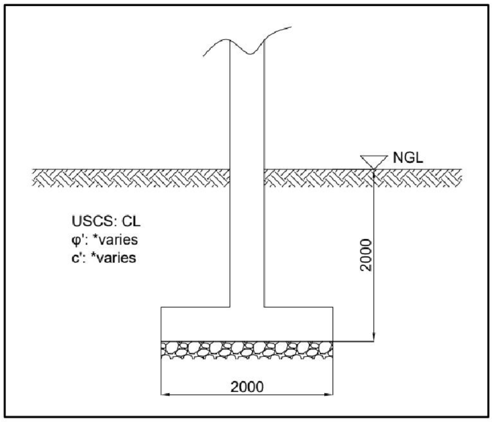

Figure 5. Idealized shallow footing foundation for bearing capacity calculations

After defining the specifics of the idealized shallow foundation, the researcher used the estimation method proposed by Sorensen & Okkels (2015) to obtain the drained angle of internal friction and drained cohesion when using both the multi-point (ϕ'_4 , c'_4) and the one-point (ϕ'_1 , c'_1) plastic limit values. The calculated shear strength parameters were used for obtaining the ultimate bearing capacities corresponding to the two different plastic limit methods (UBC_4 , UBC_1) and their corresponding allowable bearing capacities (ABC_4 , ABC_1), estimated to only be third of the ultimate bearing capacity. The ultimate and allowable bearing capacities, together with the factor of safety (FS) of (ABC_1) with respect to (UBC_4) are shown in the first part of Table 5.

The calculated allowable bearing capacities when using the plastic limit values yielded by the proposed one-point method lie only within 14.41% of the calculated allowable bearing capacities obtained by using the plastic limit values obtained via the multi-point method. As per the study conducted by Karkush *et al*. (2020) a deviation of 30% from the accepted value for the bearing capacity is still sufficient for design use. Extending the analysis in terms of factor of safety (FS), the allowable bearing capacities obtained by using the Atterberg Limits obtained from the one-point method had an average factor of safety of 3.19 when compared to the ultimate bearing capacity calculated via the use of Atterberg Limits obtained from the multi-point method. Another study on allowable bearing capacity of shallow foundation suggests that a factor of safety of at least 3.0 is required to reduce the probability of failure of a foundation on cohesive soil to acceptably small levels (Luo & Bathurst, 2017). Lastly, results of a two-tailed paired samples t-test on the calculated allowable bearing capacities suggest that there is no significant difference between the means of the ABC's for the two methods since the calculated t-value (0.6537) is less than the critical t-value (2.0096).

3.2.2 Consolidation Settlement

Since the study is based on fine-grained soils, it has been assumed that the primary consolidation undergone by the soil in the idealized scenario is magnitudes greater than the initial elastic settlement (Budhu, 2015; Das & Sobhan, 2018), and hence the elastic settlement may be neglected for the purpose of this model verification. As is the case for the bearing capacity calculations, the footing dimensions, embedment depth, and assumed soil parameters were taken from a range of measurements present in literature. For this, several studies were used (Shahin *et al*., 2002; Tezcan *et al*., 2006; Dekker & Ritsema, 1996; Mirzababaei *et al*., 2017; Skempton, 1984; Löfman & Korkiala-Tanttu, 2021). The shallow footing details are presented in Figure 6.

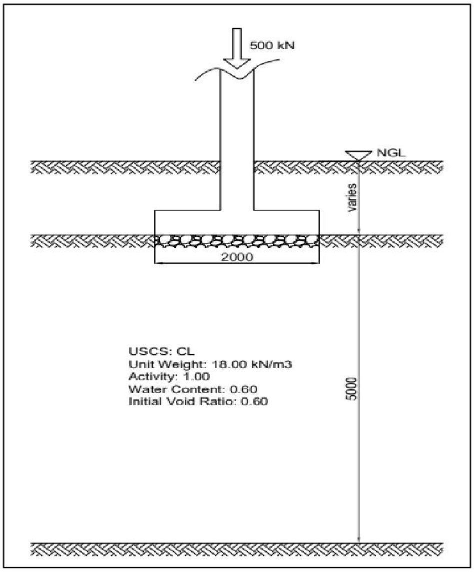

Figure 6. Idealized shallow footing foundation for settlement calculations

The empirical equation proposed by Carrier (1985) to calculate for fine grained soils was used to obtain the compression indices of using both the multi-point and one-point plastic limits (Cc_4, Cc_1), respectively. The primary consolidation settlements (Sc_4, Sc_1), corresponding to the two compression indices can then be calculated by the method proposed by Das & Sobhan (2018). The compression indices together with the primary settlements are presented in the second part of Table 5.

From the tabulated data, the average absolute percent error between the calculated consolidation settlements for both the multi-point and the one-point penetration methods (Sc_4 , Sc_1) was found to be 16.20%. This is considered more than acceptable for common engineering applications since a study suggested that a deviation of 36.1% from the true measured settlement is still acceptable (Feng & Yin, 2016). Furthermore, out of the 50 soil samples included in the study, only 15 soil samples had reduced (underestimated) calculated settlements when going from the standard multi-point method to the single-point method. Lastly, a two-tailed paired samples t-Testing reveals that there is no statistically significant difference between the settlements obtained via the one-point modified fall cone penetration method and that of its multi-point counterpart since the calculated t-value (-1.9949) is less than the critical t-value (2.0096).

Table 6. Liquefaction, USCS, and AASHTO impact assessment

No	Liquefaction		USCS		AASHTO	
	PL_4	PL_1	PL_4	PL_1	PL_4	PL_1
1	Zone C	Zone B	CL	CL	A-6	A-6
2	Zone C	Zone C	CH	CH	A-7	A-7
3	Zone C	Zone C	CH	CH	A-7	A-7
4	Zone C	Zone C	CH	CH	A-7	A-7
5	Zone C	Zone C	CH	CH	A-7	A-7
6	Zone C	Zone C	CH	CH	A-7	A-7
7	Zone C	Zone C	CL	CL	A-7	A-7
8	Zone C	Zone C	CL	CL	A-7	A-7
9	Zone C	Zone C	CH	CH	A-7	A-7
10	Zone C	Zone C	CH	CH	A-7	A-7
11	Zone C	Zone B	CL	CL	A-6	A-6
12	Zone C	Zone C	CH	CH	A-7	A-7
13	Zone C	Zone C	CH	CH	A-7	A-7
14	Zone C	Zone C	CH	CH	A-7	A-7
15	Zone C	Zone C	CH	CH	A-7	A-7
16	Zone C	Zone C	MH	MH	A-7	A-7
17	Zone C	Zone C	MH	CH	A-7	A-7
18	Zone C	Zone C	CH	CH	A-7	A-7
19	Zone C	Zone C	MH	MH	A-7	A-7

Table Continued.

20	Zone C	Zone C	MH	MH	A-7	A-7
21	Zone C	Zone C	MH	MH	A-7	A-7
22	Zone C	Zone C	MH	MH	A-7	A-7
23	Zone C	Zone C	CH	CH	A-7	A-7
24	Zone C	Zone C	MH	CH	A-7	A-7
25	Zone C	Zone C	CL	CL	A-7	A-7
26	Zone C	Zone C	MH	MH	A-7	A-7
27	Zone C	Zone C	MH	MH	A-7	A-7
28	Zone C	Zone C	CH	CH	A-7	A-7
29	Zone C	Zone C	CH	CH	A-7	A-7
30	Zone C	Zone C	CH	CH	A-7	A-7
31	Zone C	Zone C	CH	CH	A-7	A-7
32	Zone C	Zone B	CL	ML	A-7	A-7
33	Zone C	Zone C	MH	CH	A-7	A-7
34	Zone C	Zone C	CH	CH	A-7	A-7
35	Zone C	Zone C	CH	CH	A-7	A-7
36	Zone C	Zone C	MH	MH	A-7	A-7
37	Zone C	Zone C	MH	MH	A-7	A-7
38	Zone C	Zone C	CH	CH	A-7	A-7
39	Zone C	Zone C	CL	ML	A-7	A-7
40	Zone B	Zone C	CL	CL	A-6	A-6
41	Zone C	Zone C	CL	CL	A-7	A-7
42	Zone C	Zone C	CL	CL	A-7	A-7
43	Zone C	Zone C	CH	CH	A-7	A-7
44	Zone B	Zone B	CL	CL	A-6	A-6
45	Zone C	Zone C	CL	CL	A-7	A-7
46	Zone C	Zone C	CH	CH	A-7	A-7
47	Zone C	Zone C	CH	MH	A-7	A-7
48	Zone C	Zone C	CL	CL	A-7	A-7
49	Zone B	Zone C	CL	CL	A-6	A-6
50	Zone C	Zone C	CL	CL	A-7	A-7

3.2.3 Liquefaction Susceptibility

Unlike bearing capacity and settlement, liquefaction susceptibility evaluation methods relying on the Atterberg consistency limits are qualitative in nature and require less rigorous calculations if at all. To assess the impact of using the proposed one-point plastic limit test, the method proposed by Seed was utilized. Soils that fall under "Zone C" are considered not susceptible to liquefaction, while those that fall under "Zone B" require further testing to ascertain its liquefaction susceptibility and finally, those that fall under "Zone A" are considered susceptible to seismic-induced liquefaction (Youd & Idriss, 2001). The results of the liquefaction analysis performed by considering the two sets of plastic limit values are presented in the first part of Table 6.

Out of the 50 soil samples, only 5 were categorized under a different zone when (PL_1) was used instead of (PL_4). This denotes that the susceptibility evaluation method still retained 90% accuracy even when only the one-point method was utilized. This level of accuracy is more than sufficient for engineering use. This evaluation was based on a study conducted by Kongar *et al*. (2017) that reviewed different simplified shear wave methods used by practicing civil engineers for liquefaction assessment. The study surmised that the best liquefaction assessment method amongst those considered correctly predicted liquefaction only 78% of the time.

3.2.4 USCS Classification

The impact of using the plastic limit values obtained from the one-point modified fall cone penetration method in the Unified Soil Classification System (USCS) was also assessed. Since the soil particle gradation for the soils remains unchanged during the comparison, only their placement on the plasticity chart could be impacted by the difference in the plastic limits. The soil classification of the samples when using both (PL_1) and (PL_4) are presented in the second part of Table 6.

Data from Table 6 suggests that the one-point method had an 88% accuracy in classifying soils based on the USCS when compared with its multi-point counterpart. Only 6 of the 50 soil samples had a USCS classification different to the classification obtained when using the plastic limit value obtained by using the multi-point method. This level of accuracy was in line with those obtained by two independent studies who both studied alternative methods of USCS classification via CPT test results. The level of accuracy they have observed in studies were 90% and 87%, respectively (Cai *et al*., 2015; Reale *et al*., 2018). Since the accuracy obtained by the current study is in line with the previously mentioned studies, the evaluations set forth by the authors may be used. The USCS classifications produced by the plastic limit values obtained via the one-point fall cone penetration method is satisfactory for geotechnical purposes requiring soil classification.

3.2.5 AASHTO Classification

Like the impact assessment on the USCS, the impact assessment performed for the AASHTO soil classification system relied on classification of the soils based on their plasticity data. The final classifications of the soils when utilizing both (PL_1) and (PL_4) are presented in the fifth and final part of Table 6.

Based on the data tabulated, all the 50 soil samples retained the same AASHTO classification when going from the multi-point penetration method to the one-point penetration method, thereby having an accuracy of 100%. Studies on alternative methods of AASHTO classification had prediction accuracies of 92.6% and 94.35%, respectively (de Souza *et al.*, 2020; Ribeiro *et al.*, 2015). Based on their evaluation of their results, the impact of using the one-point penetration method's plastic limit values as an alternative to those yielded by the multi-point penetration method for use in classifying soils via the AASHTO soil classification system is none to minimal.

3.2.6 Testing Time and Sample Amount

To verify the testing time and sample amount savings afforded by the new method, the researcher performed both the standard multi-point and proposed one-point modified fall cone penetration method on 10 new soil samples not used in the derivation of the study's model to quantify the time and amount of sample savings going from multipoint to single-point method. The results are shown in Tables 7 and 8 for the testing times and sample amounts, respectively.

Table 7. Testing time comparison

Trial No.	Testing Time (H:M:S)		Time Savings	
	Multi-point	One-point	(H:M:S)	(%)
1	01:12:24	00:23:56	00:48:28	66.94
2	00:58:57	00:26:12	00:32:45	55.56
3	01:02:19	00:16:21	00:45:58	73.76
4	01:03:43	00:26:48	00:36:55	57.94
5	00:54:02	00:20:08	00:33:54	62.74
6	00:55:50	00:22:52	00:32:58	59.04
7	01:15:30	00:17:43	00:57:47	76.53
8	00:59:10	00:21:20	00:37:50	63.94
9	01:07:41	00:24:23	00:43:18	63.97
10	01:02:21	00:17:17	00:45:04	72.28
Average	01:03:12	00:21:42	00:41:30	65.27

From the data presented in Table 7, it has been determined that on average, the multi-point modified fall cone penetration method requires an hour and 3 minutes to perform while its single-point counterpart proposed by the researcher takes only around 22 minutes to perform. This translates to an average testing time savings of 42 minutes or 65.08% of the total multi-point method testing time. A statistical t-test performed on the testing time (converted to seconds) required for both methods was also performed. From the results of two-tailed paired samples t-testing, the researcher has determined that there is indeed a statistically significant difference between the means of the testing times of the one-point and multi-point penetration methods. This is due to the calculated t-value (18.3112) being significantly more than the critical t-value (2.2622).

Table 8. Sample amount comparison

Trial No.	Mass of Used Sample (g)		Mass Savings	
	Multi-point	One-point	(g)	(%)
1	258.7	126.7	132.0	51.02
2	304.5	130.4	174.1	57.18
3	263.3	123.4	139.9	53.13
4	244.9	115.9	129.0	52.67
5	250.1	149.5	100.6	40.22
6	232.5	138.3	94.2	40.52
7	221.4	114.5	106.9	48.28
8	256.4	149.2	107.2	41.81
9	234.1	117.9	116.2	49.64
10	225.9	132.2	93.7	41.48
Average	249.2	129.8	119.4	47.60

Data from Table 8 on the other hand details that on average, the amount of soil sample to perform the multi-point fall cone penetration test is 249 grams. Meanwhile, to perform the single-point method it is only required to have around 130 grams of dry soil sample. The total sample amount savings afforded by the proposed method may then be calculated to be an average of 119 grams per test or 47.79% when taken as a percentage of the required sample for the multi-point test. Like the analysis performed for the time savings, paired samples t-testing was also performed on the amount of soil sample used by both methods. From the results of two-tailed paired samples t-testing, it is evident that there does indeed exist a statistically significant difference between the sample amounts required to perform the one-point and the multipoint penetration methods. This is because the calculated t-value (18.3112) is more than the critical t-value (2.2622).

In actual laboratory application, the sample mass savings of the newly developed one-point method would have a negligible impact on testing cost. A difference of around 100 g does not necessitate a vastly different mode of transport. Besides, samples are also usually brought back to the laboratory together with the drilling rigs which are loaded onto trucks. The time savings afforded by the one-point method however would have a huge impact on testing cost. At the average testing speed of the multi-point method, a skilled laboratory technician working 8 hours a day would only be able to finish a maximum of 7 samples. On the other hand, if the one-point method would be utilized, the same technician would then be able to finish testing a maximum of a maximum of 21 samples in 1 day. This therefore translates into an approximately 2/3 reduction in labor cost for plastic limit testing.

4. Conclusion and Recommendation

From analyzing the multi-point modified fall cone penetration plastic limit test data of 50 fine-grained soil samples, the researcher was able to develop an equation that made it possible to obtain the plastic limit with just one instance of penetration. Furthermore, the one-point modified fall cone penetration method for plastic limit determination based on the model equation (Equation 14) was verified to be able to obtain plastic limit values with a strong positive correlation, large amount of explained variation, and no statistically significant difference to the plastic limit values otherwise obtained by using the multi-point penetration method.

Additionally, the new method was found to be applicable to soils existing outside the original 50 samples used to generate it. It was able to obtain plastic limit values with a strong positive correlation, large amount of explained variation, and no statistically significant difference to the plastic limit values otherwise obtained by using the multi-point penetration method.

The researcher was also able to demonstrate that the plastic limit values produced may be used to calculate or evaluate soils' allowable bearing capacities (ABC) sufficient for design use and with an acceptably small probability of failure, consolidation (Sc) settlements acceptable for common engineering applications, liquefaction susceptibility evaluations sufficient for engineering use, USCS classifications considered satisfactory for geotechnical engineering purposes, and AASHTO classifications with minimal to no deviation from the considered "real" classifications.

Lastly, it was demonstrated that the proposed method required significantly less testing time and amount of soil sample obtained from field testing to be performed.

The researcher then concludes that the proposed one-point plastic limit test via a modified fall cone penetration method is a viable alternative to the currently used multi-point modified fall cone penetration test that retains the innate repeatability afforded by the fall cone penetration method whilst retaining a reasonable degree of accuracy for the obtained values of soils' plastic limits and simultaneously reducing the testing time and amount of sample required to perform the plastic limit test.

For projects that have tight schedules, the significant reduction in testing time would allow contractors to deliver geotechnical reports faster. This reduction in the turnaround time of reports would allow structural engineers to work on their design earlier, indirectly propelling the whole project's schedule forward. Additionally, for soils that typically have low recovery rates (i.e. silts) wherein the samples obtained from boreholes are usually too little to be subjected to traditional testing methods, the proposed one-point method offers a viable alternative to obtain site-specific data instead of relying on correlations.

Moreover, the researcher recommends that additional studies targeted on the development of a single-point plastic limit test via the modified fall cone penetrometer address the following points:

- Viability of and possible modifications to the proposed model in calculating the Plastic Limit of soils obtained outside the geographic boundaries of Mindanao

- Development of a single-point plastic limit test via a fall cone penetrometer for coarse grained soils, specifically sandy soils

The researcher also recommends that if further studies are conducted on this proposed method, a refinement to the assumed value of (A_{ave}) be investigated. Correlations of sample-specific index properties such as the liquid limit (LL) and the percent passing U.S. standard sieve No. 200 $(F < 200)$ to obtain a sample-specific value of (A_{ave}) may further increase the accuracy of the single-point plastic limit test model. Machine learning may even be explored to aid in inferring relationships between the aforementioned variables that may otherwise be invisible to more traditional regression methods.

Finally, since the method involves only a single instance of testing for the plastic limit, there is a huge potential for developing machines that can automate the Atterberg limit testing process. Development of an automated apparatus that tests for both the liquid and plastic limits of a soil sample using the already existing one-point method for liquid limit and the newly developed one-point method for plastic limit presented in this paper may be explored.

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