Site Characterization & Economization through Pressuremeter Test: A Case Study in KATNI, M.P.

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Abstract. Katni in Madhya Pradesh is proposed to have a railway elevated corridor spanning across 33km, longer than any other in India. The geotechnical investigation spanning over two stages showed variations in SPT N value at deeper depths. The pile lengths derived from correlation with SPT N showed lot of variation. Hence an effort has been made to characterize the site through in-situ shear strength properties by conducting pressuremeter test. The variation of undrained shear strength (S_u) derived from SPT 'N' correlations and pressuremeter test is presented along with the site Np value. S_u derived from the pressuremeter test is much higher and appropriate than those derived from UU-triaxial tests and SPT 'N' correlations. The variation of S_u derived from pressuremeter tests, UU-triaxial test and SPT 'N' correlation is illustrated in this paper. Through in-situ testing, economization and standardization of foundation design was possible in this project.

Keywords: SPT N, In-situ, Pressuremeter test, Undrained shear strength, economization

1 Introduction

Katni is a major junction of West Central railways, with heavy railway traffic meeting at the junction. The freight and passenger traffic is thereby planned to be decongested through the implementation of Katni Grade Separator project.

As a part of geotechnical investigation, boreholes were carried out at different intervals throughout the project stretch. The borehole investigation showed wide variation of SPT N value leading to a disparity in the pile lengths across the stretch. Pressure meter test (PMT) which is a specialized in-situ test was carried out at various locations across the project stretch to characterize the site with respect to shear strength. The aim of this paper is to illustrate the variation of undrained shear strength (S_u) observed after analysis of PMT. The observed shear strength values are then compared with the undrained shear strength of soil correlated through SPT N and found to be higher and more consistent across the stretch. The reason for this can be stated that, the correlations are developed for geological conditions not inherent to local geology in present case study. In this paper we will observe that cohesion values ranged from 400-600kPa at deeper depths (i.e. greater than 15m) in hard residual clay through analysis of PMT data across the stretch. Through the analysis of PMT an effort was also made in this paper to characterize the site Np value which can be used to directly assess the undrained shear strength from limit pressure and initial stress of soil.

This paper also deals with on how the pile lengths can be reduced based on in-situ test rather than relying on correlations helping in economizing the project.

2 Project Description

KATNI is located in the central part of India, in state of Madhya Pradesh (M.P.). It is the largest railway junction of M.P. and 2nd busiest railway junction of India connect more than 34 major cities of India. Approximately 342 passenger trains and 300 goods trains passes the junction every day. So, to address the easy moving of traffic a grade separator in UP and DOWN line having a total length of 33KM (in total) of which 12KM is on viaduct, 4KM on retaining wall and remaining on the earthen embankment has been proposed. Based on the rail formation level and ground level difference the grade separator is divided in to three parts; Viaduct, retaining wall and embankment. There are in total 645 supports for the viaduct portion along the alignment in both the lines. A U-trough type retaining wall is proposed for a smooth grade change from embankment to viaduct.

3 Background Study

3.1 Geotechnical investigation

The project corridor being 33KM in total, the geotechnical investigation was planned in different stages. Stage 1 comprised of boreholes being carried out at 450metres spacing for getting basic BOQ in order to float the tender. Later as a part of detailed design work boreholes were carried out at a closer spacing of 75metres i.e. at every third pier location to get a detailed understanding of the soil profile in terms of classification and its origin. In total 200 boreholes were carried out including both stages. The whole borehole investigation was divided into two agencies.

The geological profile can be classified as a residual soil formation with limestone and lateritic bauxite as the parent material overlain by a shallow deposit of alluvial soil. The residual soil is in clay formation with intermediate to high plastic characteristics. In the project stretch around the Katni River, Limestone rock was encountered from shallow depth. The core recovery and rock quality designation recorded during investigation was observed to vary from 10-40% and 0-10% respectively.

3.2 Geology

The site is a mining area having major physiographic units of Vindhyan Plateau and valleys of Bhitrigarh ranges. The limestone and latertic bauxite formations are the predominant geological formations in current site. The geological profile consists of residual soil of limestone and bauxitic origin overlain by a top layer of alluvium soil. The residual soil is mostly high to medium plastic clay in nature. Limestone rock was encountered in highly weathered state in and around the Katni river area.

3.3 Soil profile:

Field exploration and laboratory tests from stage 1 and stage 2 investigations of 200 boreholes up to 40m depth shows the presence of brownish to grey color silty clay in medium stiff state at surface to hard at deeper depths. In **Fig. 1** generalized soil profile, Atterberg limits and SPT 'N' are shown at borehole locations where PMT was executed. The soil at deeper depth mostly after 10m depth is found to be intermediate plastic clay in hard state, occasionally with gravel content varying from 12 to 20%. The clay with lime content (in residual soil form) is also located at depths greater than 15m. Coarse sand with gravel content is also found as pockets at some depths.

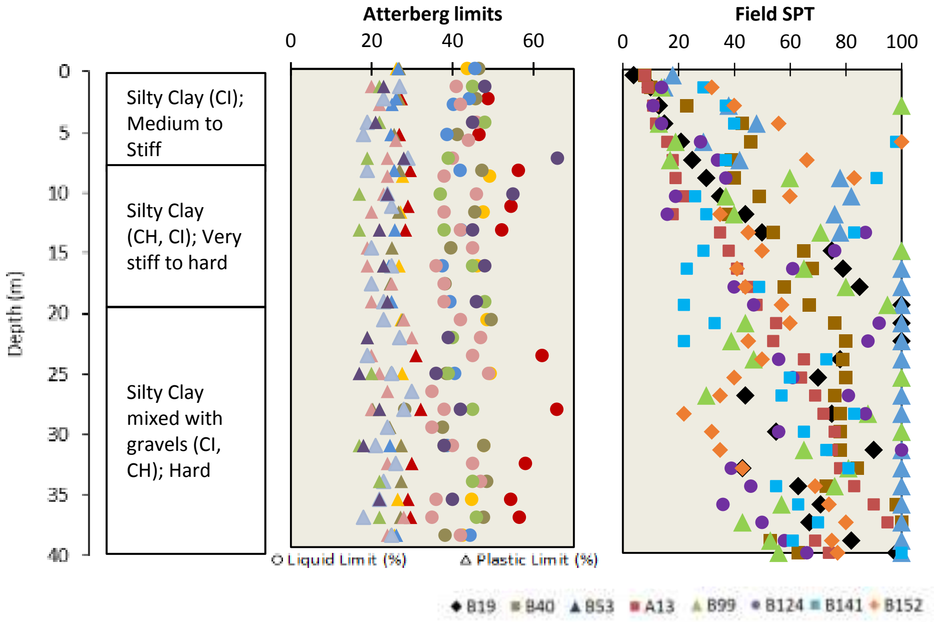
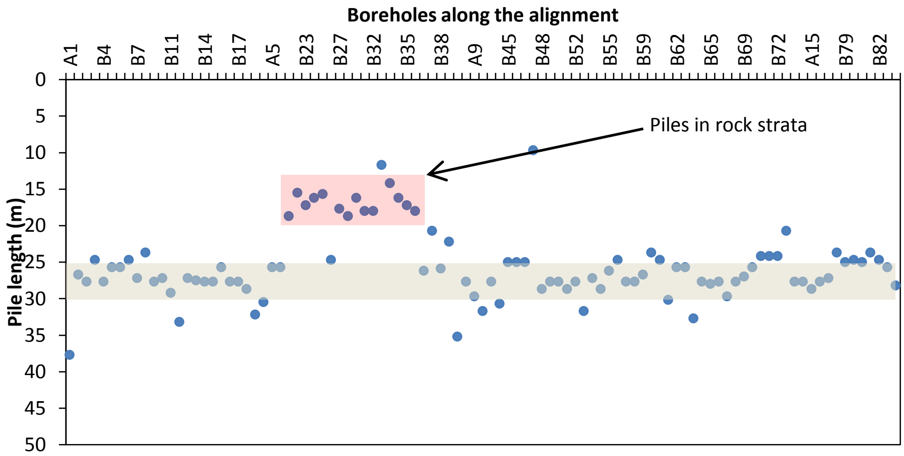


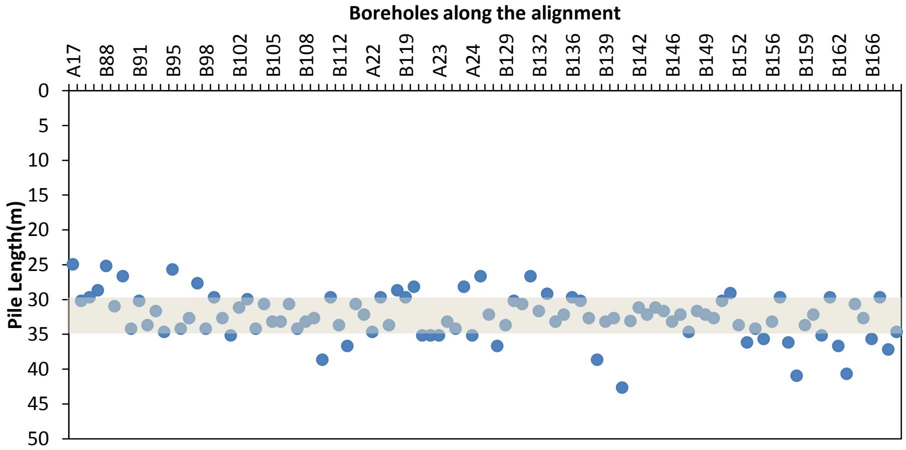
Fig. 1. Soil profile and parameters at Pressuremeter locations

3.4 Foundation recommendations

Pile was recommended as foundation for the pier structures owing to the high loads transferred from superstructure and also due to space constraints given the project runs parallel to existing railway line and urban area. The undrained shear strength (S_u) where undisturbed sampling was not possible was calculated through correlations from SPT 'N'. The pile lengths were found to vary from 25m-35m across the project stretch owing to variation in SPT 'N' values. Around katni river location the pile length vary from 18m-20m length where rock was encountered. The variation in SPT 'N' values can be attributed to different agencies using different tools for carrying out drilling as well for execution of SPT. As a result a lot of variation was observed in pile lengths for the similar soil characteristics. This led to the necessity of an in-situ Pressuremeter test to directly assess the undrained shear strength of soil strata instead of correlations developed in different geological conditions not inherent to local geology. The variation of pile length in the project stretch across the project is shown in **Fig. 2**.



(a)



(b)

Fig. 2. Variation of pile length along the alignment (a) starting from Ch. 0KM to 7.5KM (b) from 7.5KM to 14KM (approx...)

4 Pressuremeter Test

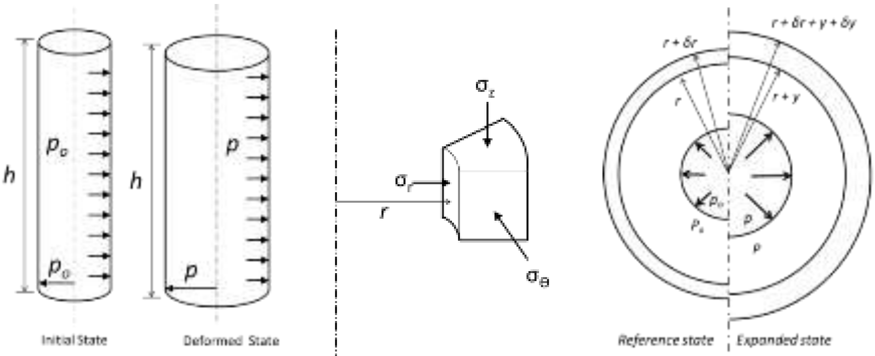
4.1 Planning

Pressuremeter test is planned in 8 boreholes, at 4 test depths in each; total 32 tests. The pressuremeter is having an outer dia. of 74mm and length of 500mm. The maximum volume and pressure capacities are 800cm^3 and 80bar respectively. The test locations and depths were decided by considering the soil\lithological profile from stage-1 & 2 investigations. As collecting undisturbed sample at depth more than 10m is found critical, the pressuremeter test is adopted for strength characterization of the deeper strata.

4.2 Analysis

Fig. 3 shows expansion if an ideal cavity in cohesive soil. The test is assumed as undrained because the duration of test is very small as compared to the drainage time which is going to occur in the drainage path of very large in order of the borehole dia. (Palmer 1972). When the borehole is drilled, the radial stress reduced to zero on the borehole wall. The soil will behave elastically if the disturbance caused due to drill is nominal. During the test, when the probe membrane touches the borehole wall the soil behaves an elastic trend in pressure-volume curve till strain softening stage. The volume (V_0) at which the curve became linear is equal to the difference between the volume of the hole and the initial volume of the probe. The reference pressure (P_0) and volume (V_0) is taken from the starting point of the linear portion. The expression for shear modulus for the expansion of a cylindrical cavity is given by Gibson and Anderson (1961):

$$G = (V_i + V_m)\frac{\Delta p}{\Delta v} \tag{1}$$



(a) (b) (c)

Fig. 3. Ideal expansion of cavity (a) expansion of cylindrical cavity (b) Element of soil in cylindrical symmetry (c) reference and expanded stage of cavity at a radius r

Where, G is the shear modulus of the soil, V_i is the initial volume of the probe, V_m is the mean volume of the probe in the elastic region, Δp is the corrected pressure increase in elastic region, Δv is the corrected volume increase in the elastic region for the increment of Δp pressure.

In the analysis of expansion of cylindrical cavity in elastic-perfectly plastic soil the reference state is taken as original in-situ state before drilling the borehole at test depth location; which can be acquired by pushing the deformed borehole wall to the original state. This could be observed in the pressure-volume curve of the pressuremeter test. When the pressuremeter probe wall touches the borehole wall the soil responds elastically. The initial point from the linear portion of the pressuremeter curve is considered as the horizontal in-situ stress. All volume is normalized with this reference volume (V_0) and volumetric strains were calculated from this volume.

According to Palmer (1972) the shear strength of the soil from pressuremeter can be derived from the slope of P: ln ($\Delta V/V$) curve. The slope of elastic and plastic phase gives peak and ultimate strength of the surrounding soil respectively. For a proper development of plastic phase the applied strain should be large enough to yield the surrounding soil annulus. So the undrained strength of the soil is given by:

$$S_u = \frac{dp}{d\left(\ln\left(\frac{\Delta V}{V}\right)\right)} \tag{2}$$

As cavity pressure increases, in plastic stage the cavity strain increases continuously with a small increment of pressure. The pressure at which the change in volume equals the cavity volume is termed as limit pressure (P_L). The expression for limit pressure is:

$$P_L = P_0 + S_u \left(1 + \ln\left(\frac{G}{S_u}\right)\right) \tag{3}$$

$$N_P = 1 + \ln\left(\frac{G}{S_u}\right) \tag{4}$$

$$S_u = \frac{P_L - P_0}{N_P} \tag{5}$$

The limit pressure depends strongly on undrained shear strength (S_u) and shear modulus (G) (Menard 1975). Marsland and Randolph (1977) proposed this method for estimation of S_u from P_L. S_u can be derived from Eq. (5) by successive estimation of S_u by equating both sides of equation. Marsland and Randolph (1977) conducted a series of plate load test and pressuremeter test and compared the results. By comparing both the in-situ test results suggested N_p value of 6.18 for London clay. Mair and Wood (1987) suggested the variation of N_P with G/S_u presented in **Fig. 4**. The variation of N_P corresponding to G/S_u is presented in **Fig. 4**. In this study the N_P value observed is in range of 4.25 to 5.26. From the figure it could be observed that N_p values are less than the values observed by Marsland and Randolph (1977) but good agreement with Mair and Wood (1987).

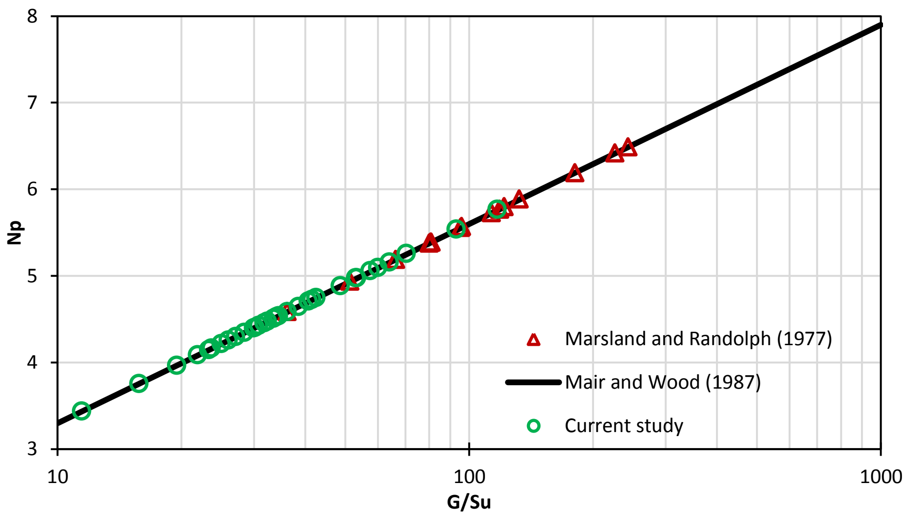


Fig. 4. Variation of pressuremeter constant with shear modulus/undrained shear strength

5 Site Characterization

5.1 Undrained shear strength (S_u)

The undrained cohesion values obtained from the analysis presented in the preceding section is shown in **Fig. 5** below. From figure it can be observed that undrained shear strength (S_u) of soil at 10-15m below ground level (GL) is reported to vary between 200-300kPa with few values showing higher S_u of 500-600kPa because of fewer readings in the plastic phase of the test. From 15-20m below GL is reported to vary between 400-500kPa and that between 20-30m below GL the cohesion values is varying between 450-600kPa.

Cole and Stroud (1976) reported S_u ranging from 400-600kPa for hard clay or very weak rock. IS 2911 Part 1 Sec 2 represents the same table given by Cole and Stroud (1976), but recommends the table to be used only for judgement of shear strength properties of weathered rocks.

5.2 Comparison of undrained cohesion values

The initial estimation of undrained shear strength of soil for foundation recommendation was carried out in this project through correlations from SPT 'N'. At later stage due to variation of SPT 'N' values leading to variation in pile length, the in-situ shear strength of soil was analyzed through Pressuremeter test.

In this section a comparison of cohesion values obtained from SPT 'N' with the cohesion values obtained from Pressuremeter test is illustrated. From **Fig. 6** it can be observed that the range of undrained shear strength (Su) values obtained from correlations varies from 240kPa-340kPa. The in-situ undrained shear strength values obtained from Pressuremeter test at similar depth is observed to be higher. The reason for this may be explained in the shortcomings of the correlations developed in soil conditions not same as local geology. Moreover the correlations being based on SPT 'N', there is uncertainty about the efficiency of the hammer used at project site and that used for developing the correlation.

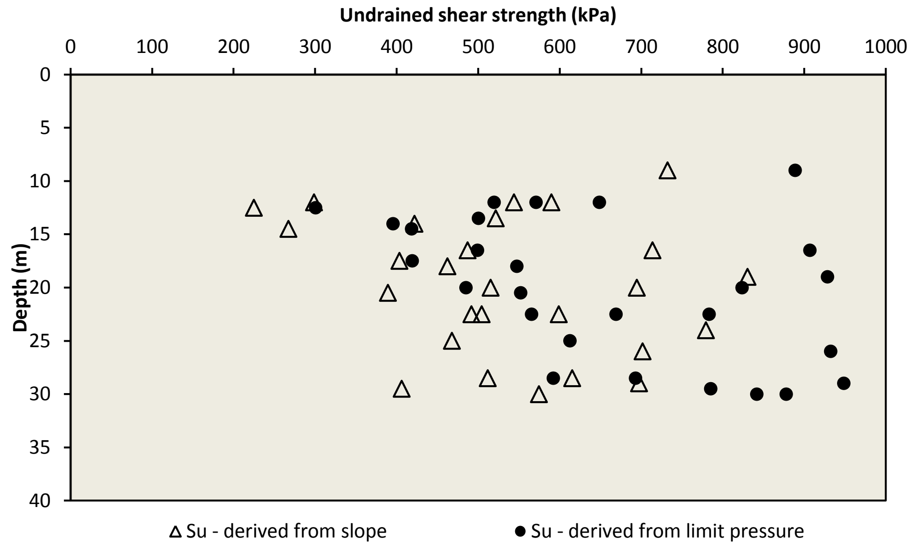


Fig. 5. Undrained shear strength derived from pressuremeter test

A comparison can also be drawn with the undrained shear strength (S_u) obtained from UU- Triaxial Tests (on UDS); with some amount of limitation as the numbers of UU- Triaxial tests for comparison are less. It can be observed that at depth of 10-15m from GL the UU- Triaxial test gives S_u of 140-240kPa whereas the same from Pressuremeter test is reported as 210-300kPa. In deeper strata, at same location the difference between S_u, derived from UU-Triaxial test and pressuremeter tests increases. This can be observed from **Fig. 6**, where UU- Triaxial test report S_u in the range of 250 kPa whereas in Pressuremeter test it is of 400kPa. The reason is explained by many previous researchers Hansen & Gibson (1949); Wood & Worth (1977). According to Hansen & Gibson (1949) the shear strength mobilized on a vertical plane as in Pressuremeter test is much greater than; the shear strength mobilized a plane inclined 45 deg. to the horizontal in Triaxial compression test; because the lateral earth pressure applied in Pressuremeter test in in-situ condition is very high than the confined pressure applied in Triaxial compression test. Wood and Worth (1977) stated that on the expansion of cavity the radial stress σ_r becomes the major principal stress (more than existing vertical stress σ_z) and σ_Θ becomes minor principal stress. Hence, these two principal stresses causing the failure on a vertical plane. So the vertical stress σ_z becomes an intermediate principal stress causing no effect on the soil element.

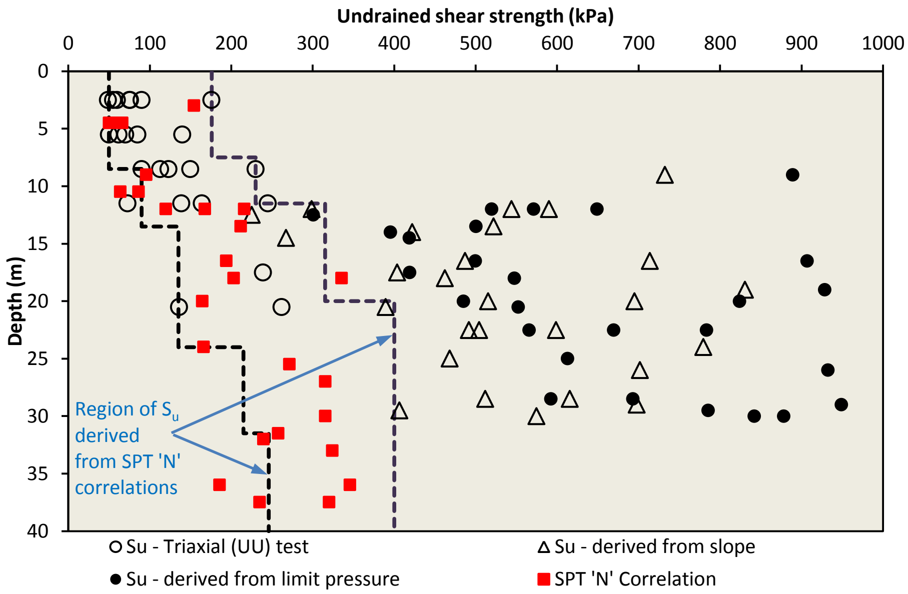


Fig. 6. Comparison of undrained shear strengths from UU-Triaxial test, SPT 'N' Correlations and derived from pressuremeter tests (from slope and limit pressure)

The correlation graph in current study and given by Stroud (1972) is shown in **Fig. 7**. The factor f_1 (S_u/N_{60}) in current study is developed by dividing S_u derived from pressuremeter test by SPT 'N' observed at same depth corrected for 60% hammer efficiency. It can be observed that the f_1 factor ranges from 7.5 to 15. However the range can be narrowed down to 7.5 to 13 by only considering those S_u which was derived from Pressuremeter tests with fully developed plastic phase. This gives some evidence that the correlations developed on soil not inherent to local geology may not give correct assessment of undrained shear strength values.

6 Discussion

6.1 Economic design

In the preceding sections, the variation in pile lengths was illustrated. Through soil classification the sub-soil profile was observed to be same across the project stretch. As a result to understand the generalized undrained shear strength of soil pressuremeter test was carried out across the project stretch at different locations. Through analysis of pressuremeter test data, the undrained shear strength of soil at different depths was assessed as illustrated in preceding section.

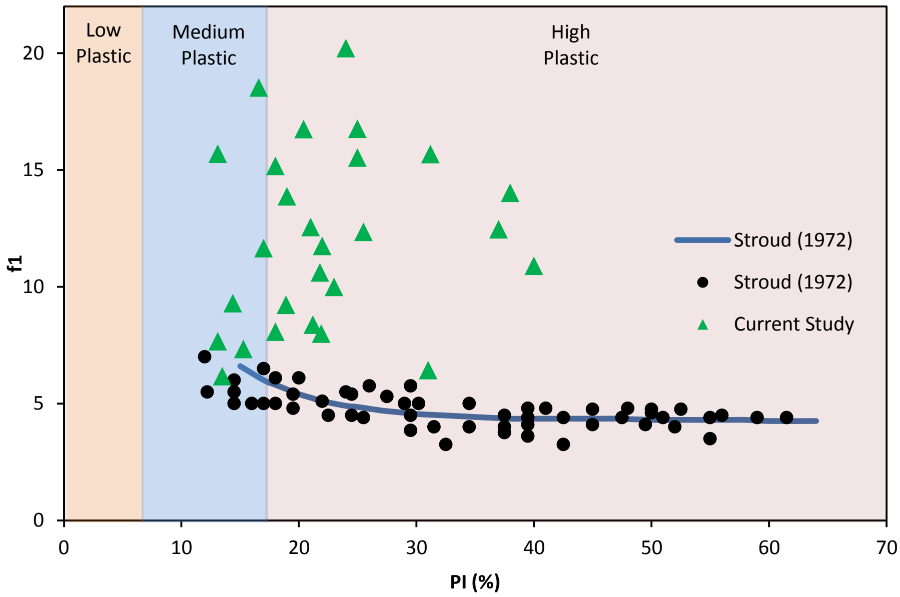


Fig. 7. Variation of f_1 values with respect to plasticity index

The pile length calculation was done based on the range of S_u data achieved through pressuremeter and was observed to vary from 23-24metres. Pile length for the 645 pier supports was reduced through undrained shear parameters analyzed from pressuremeter data resulting in economization of project. Further to this, with the variation in pile length reduced from previous recommendations, the ease of construction for Contractor in executing pile foundations for all pier supports at 26m intervals (mostly) increased manifold.

Although the advantages of PMT are many, there are few site issues that were encountered during the execution. The issues can be taken care through proper checking of all equipment and discussion with experienced people having good knowledge on equipment of PMT. However site specific issues with PMT have to be resolved through wise engineering judgement and expertise.

7 Conclusion

Katni, located in central India have stiff formations of limestone and bauxite in residual form. The soil stratum is of very stiff clay formation having high to intermediate plastic clay. The site characterization through conventional geotechnical investigation found to be difficult in current study. So, the adoption of PMT test had been done for strength characterization of the strata. The foundation had been designed based on the PMT test data obtained. Through this paper we have been able to illustrate the site based undrained shear strength values and the effects of using the same for foundation analysis in correctly judging the strength characteristics of soil. The important points that can be inferred from this paper are:

1. The in-situ shear strength parameters achieved through PMT are higher and more reliable than that analyzed through correlations.

2. The in-situ shear strength of soil through PMT is higher than shear strength observed from unconsolidated undrained triaxial test because of different confining pressure and stress paths.

3. The f_1 factor which is majorly used to correlate the undrained shear strength (S_u) of soil where undisturbed samples are not possible to be collected, grossly underestimates the shear strength of soil in the current case study. Hence the applicability of correlations for different sub-soil conditions needs to be checked.

4. The N_p value which is used to correlate cohesion value from PMT was found to vary from 4.25 to 5.26 for the current case which is slightly lower than suggested by different authors, albeit for different soil conditions.

5. Through correct assessment of undrained shear strength of soil (S_u), considerable quantity of pile length savings was made and being able to achieve generalized shear parameters helped in reduction of pile length variation.

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