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ORIGINAL PAPER

Effect of Uncertainties in the Field Load Testing on the Observed Load–Settlement Response

S. M. Dasaka · Aniruddh Jain · Y. A. Kolekar

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Abstract Field plate load test (PLT)/pile load test is being considered the most suitable technique to obtain load–settlement response of foundations resting on soil/rock. The load–settlement data obtained from PLT is routinely used in the evaluation of several design parameters of soil, such as bearing capacity, settlement of foundations, modulus of subgrade reaction, Young's modulus, etc. This information is very much essential and would assist the engineers in the design decision process. However, due to various sources of uncertainties in testing procedures, viz. lack of suitable equipment and experimental know-how, inadequate codal provisions, etc., these tests often produce unrealistic results, and create chaos in the design decisions. In this paper, a failed field PLT is discussed and various irregularities associated with the field testing are identified, viz., non-maintained load during loading stages, man-made disturbance of the influence zone during the testing, improper placement of supports to datum bar, etc. Some of the above can be easily avoided by using little common sense. However, it has been noted from several field load tests that the load on the plate seldom made constant during the loading cycles. The effect of non-maintained loading on the load–settlement behavior is systematically analysed, through a series of laboratory PLTs, simulating the field conditions. By comparing the results obtained from the laboratory and field load tests, it is observed that the bearing resistance of the soil is highly overestimated due to non-maintained load. Equilibrium pressures are attained only for lower pressure range of 100–200 kPa after 15–20 min of load application, and for higher pressure range, the pressure on the plate is continuously decreased. Based on the load–resistance factored design approach, a resistance factor of 0.5 is obtained for bearing pressure, which suggests that there is significant variability in the bearing pressure. In conclusion, proper care should be taken during the load testing, and the codes of practice should be revised from time to time.

Keywords Field practice · Codal provisions · Plate load test · Maintained load · Load–settlement response · Resistance factor

Jain Aniruddh—Formerly Department of Civil Engineering, Indian Institute of Technology Bombay.

S. M. Dasaka (✉) · A. Jain · Y. A. Kolekar
Department of Civil Engineering, Indian Institute of Technology Bombay, Mumbai 400 076, India
e-mail: dasaka@civil.iitb.ac.in

A. Jain
e-mail: aniruddh.jain@arup.com

Y. A. Kolekar
e-mail: yashwantkolekar@gmail.com

A. Jain
Arup Consulting Group, Mumbai, India

Introduction

Plate load test (PLT) or pile load test is considered as one of the most suitable techniques to obtain load–settlement (or pressure–settlement) response of foundations resting on soil/rock, due to high degree of uncertainties associated with other in situ and laboratory testing procedures, and simplified transformation models that are in routinely used. The load–settlement data obtained from PLT is routinely used in the evaluation of several design parameters of soil, such as bearing capacity, settlement of foundations, modulus of subgrade reaction, Young's modulus, etc. Load testing may be conducted either by stress-controlled technique or strain-controlled (constant rate of penetration) technique. The former technique is used to understand the drained loading characteristics of the soil, and the latter is performed to obtain the undrained loading characteristics [4]. The data on load–settlement response is very much essential during the design decision process to select a suitable foundation for a structure, taking into consideration safety, serviceability and economic aspects.

Literature

Stress-controlled load testing, also known as maintained load test, has been widely recommended to obtain the load–settlement response of soil–foundation system [1, 6, 8]. This technique was used by several researchers to predict the behavior of foundations in situ [3, 5]. Briaud and Gibbens [3] carried out field PLTs on five large square footings resting on uniform medium dense silty fine silica sand deposit with footing sizes ranging from 1 to 3 m. The study aimed at understanding the confidence levels associated with widely used methods to predict bearing capacity and settlements of footings at this site, using the data obtained from in situ and laboratory tests (viz., SPT, CPT, PMT, DMT, laboratory triaxial compression tests, etc.). Similarly, da Fonseca [5] carried out PLTs on saprolitic soil derived from granite, for comparing the measured behavior of footings with that predicted using various design procedures developed worldwide. The load testing was also used in the field and laboratory to assess the effectiveness of various ground improvement techniques, such as reinforced soil beds, stone columns, etc.

It is common practice to conduct field PLTs under reaction loading, as it is a simple loading method compared to other alternatives, such as gravity loading. In this, a series of loads are applied through a hydraulic jack working against a reaction loading in the form of a kentledge assembly/anchor piles, with a definite time lag between consecutive load increments. However, due to various sources of uncertainties in testing procedures, viz. lack of suitable equipment and experimental know-how, inadequate codal provisions, these tests often produce unrealistic results, and lead to uncertainties in the design decisions.

Mohan et al. [10] noted the difficulties involved with maintaining the load on the pile constant, and suggested an alternative approach, named as "the method of equilibrium", to avoid the problems of continuous attention and occasional pumping of the hydraulic jack, without a load maintainer. Through a series of pile load tests on precast, driven and bored cast-in situ piles, it was demonstrated that with the use of conventional hydraulic jack, a state of equilibrium of pressure and settlement, is generally found to exist, and hence suggested that a higher incremental load than the usual be applied to simulate the maintained load test, when load maintainer is not available. It was also pointed out that the time required by the method of equilibrium is generally reduced to about one-third of that required in a maintained load test.

Salient Features of Load Testing

Load in Each Stage

Indian Code of Practice on Load Tests [8] recommends that the load shall be applied in cumulative equal increments up to 1 kg/cm^2 (100 kPa) or one-fifth of the estimated ultimate bearing capacity. However, ASTM D 1194-94 [1] states that the load should be applied in cumulative equal increments of not more than 1.0 ton/ft^2 (95 kPa), or one-tenth of the estimated bearing capacity. Da Fonseca [5] conducted the load test up to a maximum load intensity of 1,000 kPa, using 35 increments of load.

Duration of Each Load Stage

In traditional methods of load tests the load applied is maintained constant either for a fixed period or until the rate of settlement diminishes to a negligible value [10]. For soils other than clayey soils, IS: 1888-2002 [8] suggests that each load increment shall be kept for not less than 1 h or up to a time when the rate of settlement gets appreciably reduced to a value of 0.02 mm/min. However, for clayey soils, the code suggests that the load should be increased to the next stage either when the curve indicates that the settlement has exceeded 70–80 % of the probable ultimate settlement at that stage or at the end of 24 h period. On the other hand, ASTM D 1194-94 [1] specifies that after the application of each load increment, the cumulative load be maintained for a selected time interval of not less than 15 min. Briaud and Gibbens [3] conducted tests with each loading stage for 30 min, whereas, da Fonseca [5] used each load for 4 h. In both the above studies, it was clearly noted that the load on the plate was maintained during each loading stage.

From the above literature, it is evident that there is no specific mention in [8] that the load in each stage be maintained. It is quite obvious that with the use of conventional hand operated hydraulic jacking system for reaction loading, it is hardly possible to maintain the load on the plate, due to the fact that there is a continuous decrease of pressure on the plate, with onset of settlement of plate. The only exceptions to this are: (i) when the operator continuously monitors the pressure gauge reading and operates the hand pump as and when required to maintain the load on the plate during each loading stage, which is hard to believe, especially when the PLT lasts for more than a couple of days, and (ii) when an automatic pressure control device (load maintainer) is attached to the pumping unit assembly.

Motivation

For reaction loading using conventional hand-operated hydraulic jack, with application of load on the plate, the plate penetrates into the soil with a corresponding reduction in the applied pressure. However, in the field, due to inadequate testing equipment, lack of technical know-how of the crew, and negligence, the condition of maintained load during loading stages is seldom satisfied. This non-maintained load certainly influences the settlement of plate corresponding to each load, and the present study focuses on this aspect. This research work is emanated from the recent experience of the first author at a power plant site, where a few field PLTs were conducted, for which he acted as an advisor.

Case Study

Selected in situ and laboratory geotechnical studies were conducted for design of a suitable foundation system to support important plant structures of a thermal power plant, such as Turbine Generator, Chimney, etc.

A few soil samples were collected from the proposed locations of the above structures, to evaluate index and other important soil properties, through laboratory studies. It was noted from the particle size analysis that the soil at the site is predominantly fine grained up to a depth of 20 m below natural ground level. The liquid and plastic limits of the soil were in the range of 37–56 and 24–28, respectively, with minimum plasticity index of 13 and maximum plasticity index of 30. Based on the Indian Standard Classification System, IS: 1498-2002 [7], the soil at the site was broadly classified as sandy clay of intermediate plasticity (group symbol: CI). The maximum observed free swell was around 30 %, and average swelling pressure was 24 kPa. The observed coefficient of consolidation of the remoulded soil was in the range of 1.6×10^{-4}–5.0×10^{-4} cm^2/s.

Field PLTs were also conducted as part of the geotechnical investigations, and they were carried out as per the specifications outlined in Indian Standard Code of Practice, IS: 1888-2002 [8]. The size of the plate selected was 0.6 m $\times$ 0.6 m, and the tests were conducted at 4 m depth below the natural ground level. No ground water table was observed in the vicinity of the plate, as the test was conducted during a dry season. The load was applied on the plate through a hydraulic jack against a heavy kentledge of sand bags, in five equal load increments. A pressure gauge was attached to the hose connecting manually operated pumping unit and the hydraulic jack to control the pressure applied on the plate. As there was no pressure regulator within the hydraulic assembly used by most of the contractors (a controller helps in maintaining the pressure on the plate constant, by pumping additional oil into the hydraulic unit), there was a drop in the pressure on the plate, as the plate settled into the soil. The rate of drop of the pressure on the plate may be directly proportional to the rate at which the plate settles into the soil. Settlement of the plate was measured using two manually recording dial gauges placed on the plate at diagonally opposite locations. Table 1 shows a typical time–settlement data recorded during the application of one of the load increments. For this loading stage, 400 kPa of cumulative pressure was applied on the plate, which corresponds to an initial pressure gauge reading of 320 kg/cm^2 ($\approx$32,000 kPa). This loading stage was continued for 12 h. The pressure gauge reading just before the application of next load increment was recorded, and it was observed that the pressure gauge reading was reduced to 250 kg/cm^2 (25,000 kPa) from its initial value of 320 kg/cm^2 ($\approx$32,000 kPa).

Figure 1 shows the time–settlement response under each load increment for one of the load tests (PLT-1). Had there been a pressure-controller based hydraulic pumping unit or a similar mechanism to maintain the pressure applied on the plate, the observed settlements under each loading stage would have been higher than were observed in the field. Lack of such an arrangement led to a flatter load–settlement response (lower slope of load–settlement curves), and may eventually be responsible for overestimating the bearing capacity and underestimating the settlement corresponding to any load on the plate.

Experimental Programme

To demonstrate and further understand the effect of non-maintained load on the load–settlement behavior of the plate, two types of laboratory PLTs are conducted on remolded marine clay, which was obtained from a site in the west coast in Mumbai. In the first test, the load at each stage is maintained constant and in the second test the load at each stage is not maintained, which very well simulate the observed load–time response of the field PLT. Some of the important engineering properties of marine clay, obtained from laboratory tests on remoulded soil samples, are presented in Table 2. The results of non-maintained and maintained load tests are presented in terms of time–settlement response.

Test Bed Preparation

The clay bed for all the tests is prepared in a testing tank of plan dimensions 46 cm $\times$ 46 cm and 41 cm depth. The

Table 1 Recorded time–settlement data for a loading stage of field plate load test

Stress on the plate (kPa)	Load on plate (kN)	Pressure gauge reading (kPa)	Time (min)	Settlements				
				Dial gauge I (LC = 0.01 mm) set at zero		Dial gauge II (LC = 0.01 mm) set at zero		Average (mm)
				Reading	Settlement at the end (mm)	Reading	Settlement at the end (mm)	
400	144	32,000	0	528		514		
			1	531		517		
			2.25	534		518		
			4	535		519		
			6.25	537		520		
			9	539		521		
			16	542		523		
			25	543		524		
			30	543		525		
			60	548		528		
			90	550		530		
			120	552		531		
			180	555		533		
			240	557		535		
			300	559		536		
			360	560		537		
			420	562		538		
			480	563		538		
			540	564		539		
			600	565		539		
			660	566		540		
			720	566	5.66	540	5.40	5.53

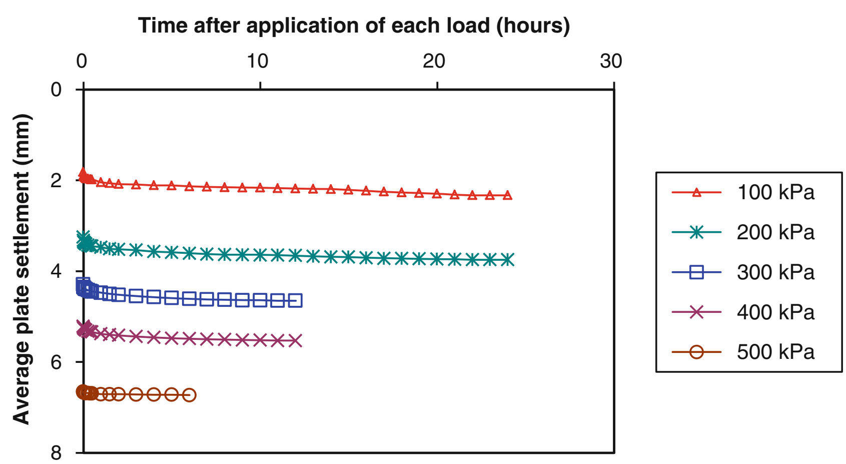

Fig. 1 Time–settlement response of in situ plate load test (PLT-1) at each load increment

inner surface of the tank is coated with a layer of metallic paint to reduce the boundary effects. Apart from this, a thin layer of oil is also applied over the painted surface. The moist soil at water content of 30 % (at its plastic limit) is placed in the tank and is compacted in layers and for each layer 9 kg of soil is used. Each layer of soil is compacted with a special hammer with equal number of blows to achieve uniform and consistent soil beds.

Table 2 Engineering properties of marine clay

Specific gravity of soil solids, G_s	2.66
Optimum moisture content	28 %
Maximum dry density	13.8 kN/m^3
Liquid limit	56
Plastic limit	30
Plasticity index	26
Shrinkage limit	11
Indian Standard Soil Classification	CH

Load Tests

For the first set of tests, i.e., maintained load tests, the tank with clay bed is mounted on the base of a specially designed reaction frame, and a lever arm based gravity loading is used to maintain the load. The loading plate used in the present study is made up of mild steel, and having a size of 10 cm × 10 cm × 0.7 cm thick. For each increment, the load is maintained till the settlement reaches 0.02 mm/min or less, following the guidelines of IS: 1888-2002 [8]. Settlements are observed with the help of two diagonally placed LVDTs at 1, 2.25, 4, 6.25, 9, 16, 25, 36, 49, 55 and 60 min from the start of the test for each load increment. A schematic view of the test setup and the reaction frame is shown in Fig. 2.

For the second set of tests, i.e., non-maintained load tests, the tank is mounted on a self-supporting reaction frame and load is applied on the plate through hand operated hydraulic jack, which is fixed to the cross beam supported at the upper portion of the reaction frame. The plate is kept at the centre of the tank, and test is conducted by applying cumulative equal increments of load on the plate. At the start of each loading stage, the hand operated lever of the hydraulic pumping unit is continuously operated till the required pressure is read in the pressure gauge. Once the required pressure is reached, the level of the pumping unit is untouched till the next stage of loading is reached. The displacements are continuously recorded with the help of LVDTs at the above mentioned time intervals. Figure 3a, b show the loading mechanism, and arrangement of LVDTs, load cell, and the loading plate.

Results and Discussion

The time versus pressure response is plotted for both the cases described in the above section and shown in Fig. 4. It is obvious that a stepped pattern of load–time response is observed when the load is maintained constant at each loading stage. For the case of non-maintained load, after application of each load increment, a gradual reduction in applied load is observed, with an associated settlement of plate. At lower pressures, viz., 100–200 kPa, the pressure on the plate is continuously decreased, and attained an equilibrium pressure at around 15–20 min after load application, in line with the observations of Mohan et al. [10]. However, at higher pressure increments, i.e., 300–500 kPa, no equilibrium pressure is noted, which contradicts the observations of Mohan et al. [10].

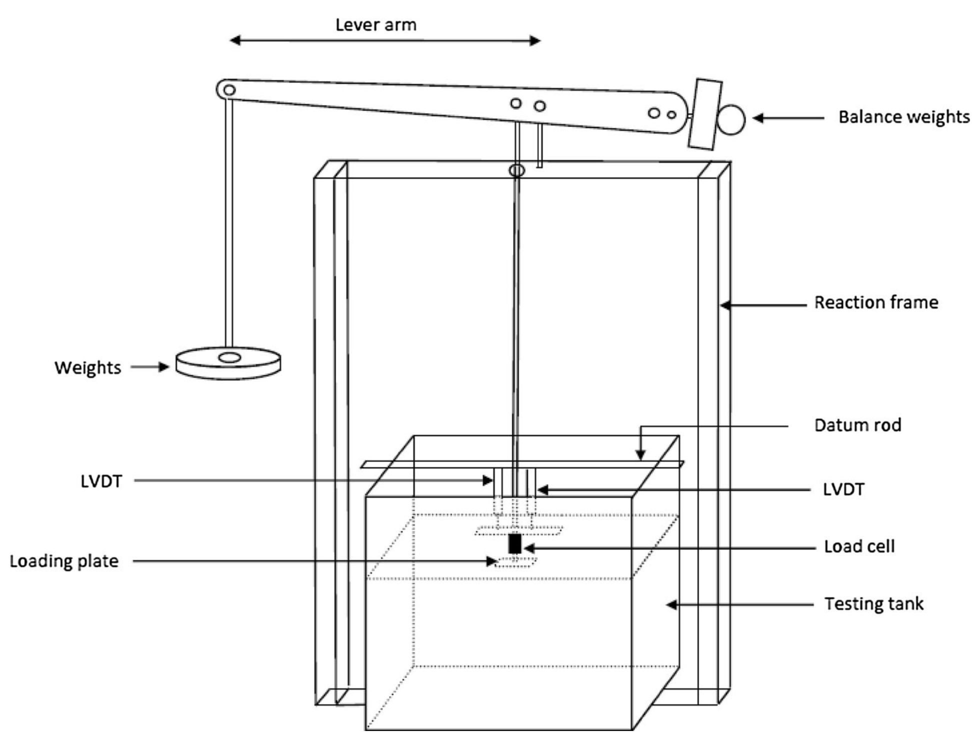

Fig. 2 Schematic diagram of laboratory plate load test set-up with lever arm mechanism for maintained loading

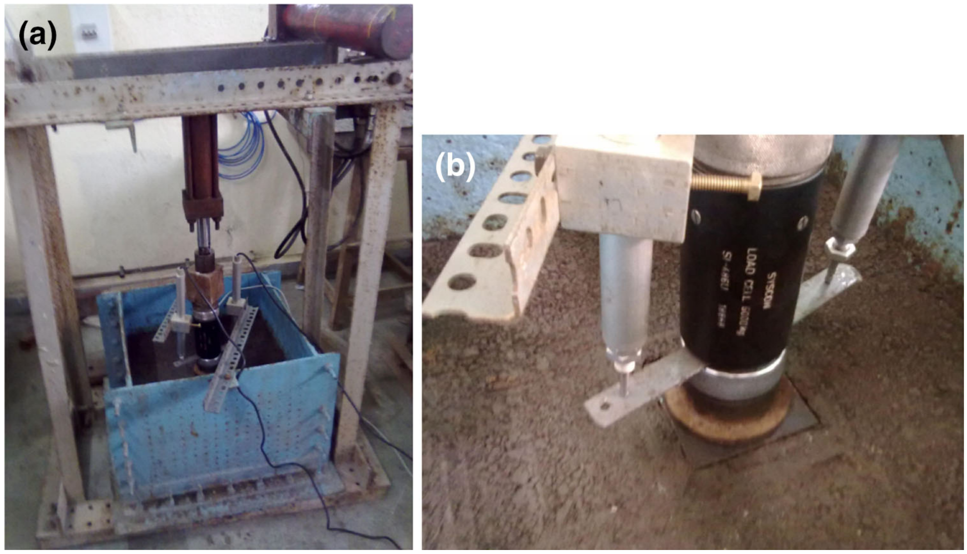

Fig. 3 **a** Laboratory plate load test setup with hydraulic jack-reaction load mechanism for non-maintained loading, **b** detailed view of the load cell and plate arrangement

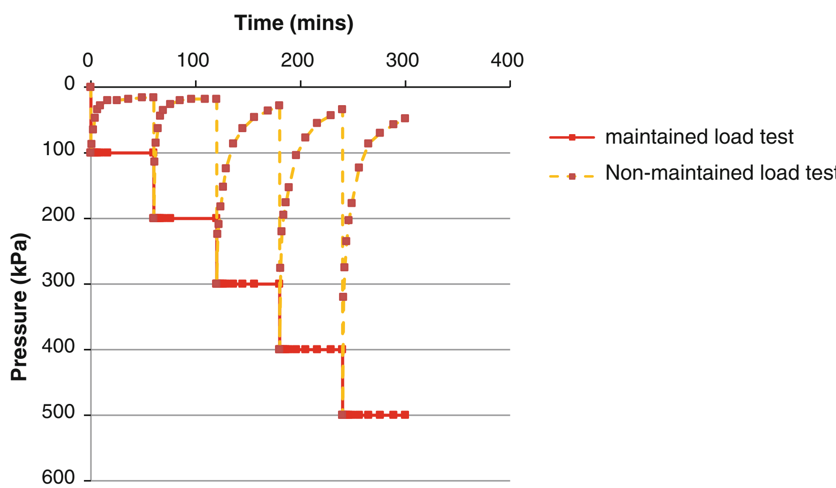

Fig. 4 Observed pressure variation on the plate during the laboratory load testing

From the time versus settlement response shown in Fig. 5, it is observed that for the case of maintained loading, settlements increase slowly and finally reduce to a rate less than 0.02 mm/min. Whereas, for non-maintained load case, after a high initial settlement, a rebound is observed, which gradually increased with time and finally attained a near constant value.

Pressure–settlement response obtained from laboratory PLT on marine clay is shown in Fig. 6, for both maintained and non-maintained load cases. It is clearly seen from the figure that the pressure–settlement response for both the cases are quite different, with non-maintained load test exhibiting flatter pressure–settlement response (or lower slope of pressure–settlement curves). The test results are highly repeatable, as shown in Fig. 7. Table 3 shows the variation of pressures at various settlement levels, for both maintained and non-maintained load cases. The bias factor, which is defined as the ratio of pressures obtained from non-maintained and maintained load tests for various settlement levels varies between 0.62 and 0.67, with an average bias factor of 0.65. Had there been a servo-controller based hydraulic pumping unit (load maintainer) or a similar mechanism to maintain the pressure on the plate during each loading cycle, the observed settlements under each loading stage would have been still higher than were observed in the present study. This kind of discrepancies in the load–settlement (or pressure–settlement) response obtained from a PLT can be avoided if proper care is exercised while load testing, and giving due attention to the codal provisions. It is also important that review of the relevant codes of practice be taken up from time to time in view of the above mentioned inconsistencies, and revised, for providing guidelines for the benefit of the geotechnical engineering community. Further studies in this direction are warranted, which may substantiate the present research findings.

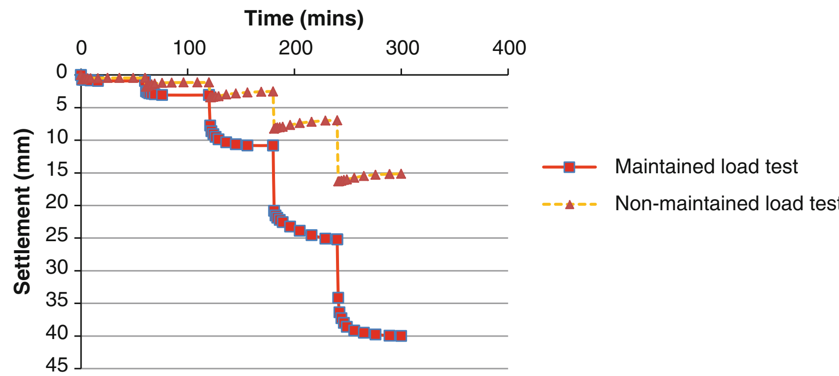

Fig. 5 Effect of maintained and non-maintained load on the laboratory plate settlement profile

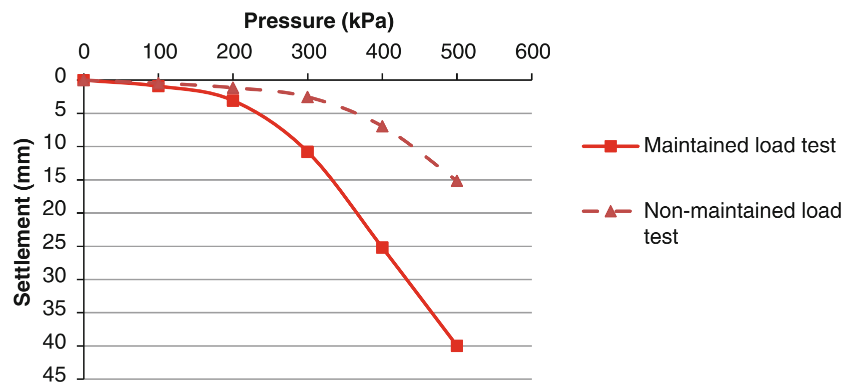

Fig. 6 Observed pressure–settlement response obtained from laboratory plate load test

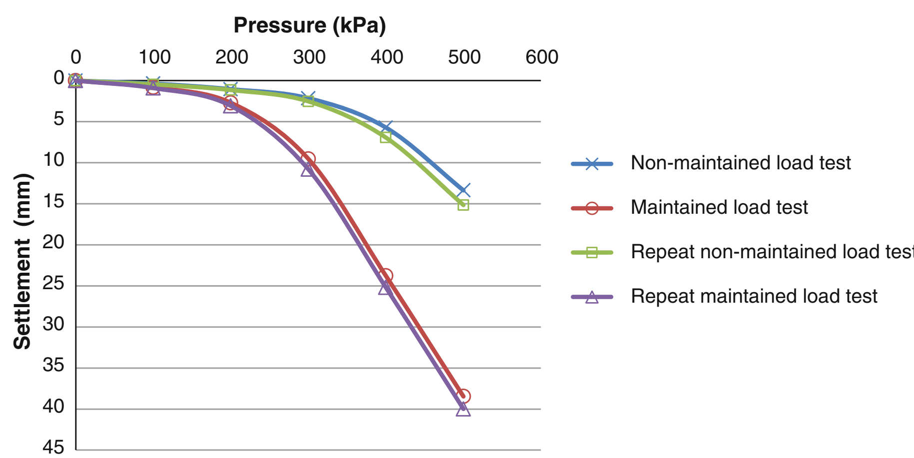

Fig. 7 Repeatability of pressure–settlement response obtained from laboratory plate load test

Measured and Predicted Load–Settlement Behaviour

It is not very uncommon to the geotechnical community to witness the predicted load–settlement behaviour deviating from the measured data. From the analysis of load–settlement data of five large footings [3, 9], it was pointed out that the measured pressure on these footings seldom matches with the corresponding predicted values (Fig. 8). One can notice a wide variation in the predictions, which can be attributed to several uncertainties involved in the geotechnical design and decision processes. Most of the methods underpredict or provide very conservative estimates of bearing pressures. Similarly, the deviation of bearing pressures obtained from maintained and non-maintained load tests is plotted in Fig. 9. As reported in the previous section, the non-maintained load tests overpredict the bearing pressures, and may lead to unsafe designs. The mean bias factor, defined as the ratio of the average of measured bearing pressures and the corresponding predicted bearing pressures for all the five footings, is plotted in Fig. 10, for the data discussed in Fig. 8. It can be noticed that the mean bias factors vary as low as 0.6 to as high as 4.6.

Table 3 Pressures at various settlement levels for both maintained and non-maintained laboratory load tests

Settlements (mm)	Pressure (non-maintained load test), kPa	Pressure (maintained load test), kPa	Bias factor (ratio of pressure from maintained test and pressure from non-maintained test)
2	290	180	0.62
4	360	225	0.63
6	405	260	0.64
8	435	285	0.66
10	460	302	0.66
12	482	321	0.67
15.15	500	331	0.66
Average bias factor			0.65

Load–Resistance Factored Design (LRFD)

It is obvious that all the geotechnical data, whether obtained from in situ or laboratory tests, exhibit some degree of variability, and routinely this variability in the design data is addressed by using a suitable factor of safety in the designs with an intention to avoid geotechnical failures. Higher variability would certainly require greater factor of safety. However, the factors of safety reported in the codes of practice are generally based on the experience, without explicitly considering of the variability associated with various parameters involved in the designs, and hence there exists lot of subjectivity with these values, design decisions may many times. Probability based design, which facilitates explicit consideration of the variability associated with various parameters involved in the designs, can be effectively used to provide satisfactory designs. However, complete probabilistic based designs cannot be used in routine design decision process, as they require lots of data on loads and geotechnical parameters (or resistance), such as, coefficient of variation, probability distribution, spatial variation, etc. and involve rigorous mathematical computations.

A trade-off between the conventional factor of safety based design and rigorous probabilistic based design is the implementation of Limit State Design approach. In this approach, load and resistance factors are derived on regional basis, satisfying the basic design criteria that during the design life of the structure, the sum of the factored resistance should be more than or equal to the sum of the factored load component, for an acceptable probability of failure [2], as shown below. Figure 11 shows the general philosophy of Limit State Design.

$$\Phi R_n = \sum \alpha_i S_{ni} \tag{1}$$

where Φ and α are resistance and load factors, and R_n and S_n are nominal resistance and load components, and i refers to various load components to which a foundation is subjected, such as dead load, live load, etc. Nominal resistance and nominal load are related to their respective mean values as:

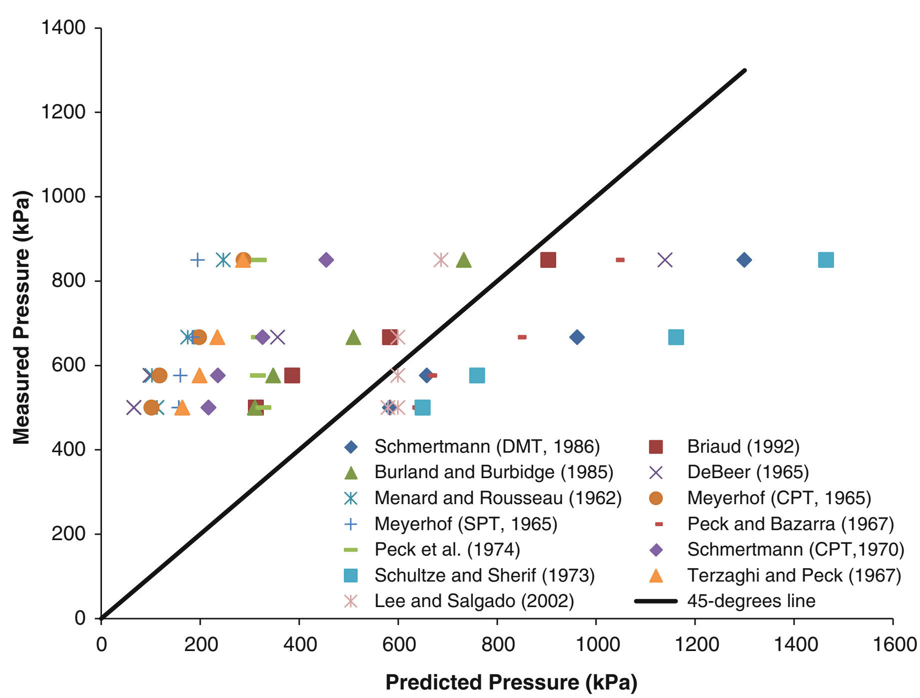

Fig. 8 Measured and predicted bearing pressures of five large size footings at 25 mm footing settlement

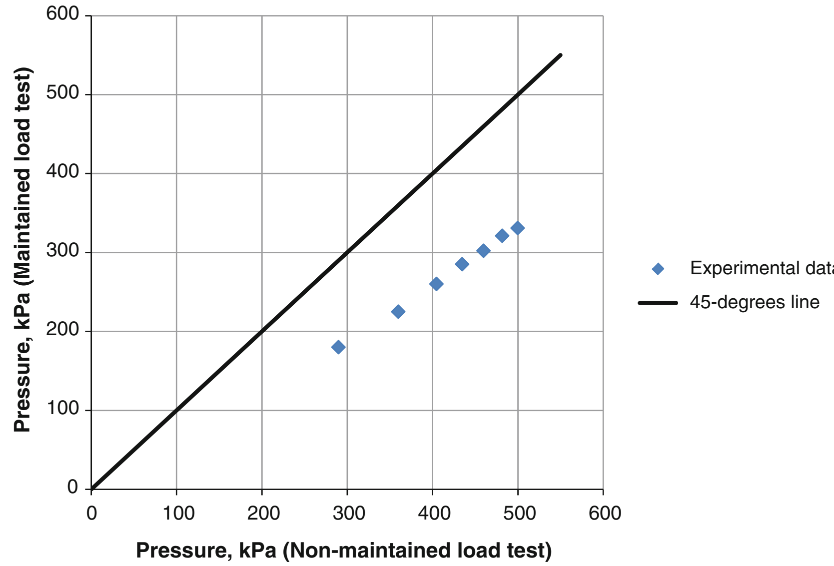

Fig. 9 Deviation of bearing pressures of model footing obtained from maintained and non-maintained load tests

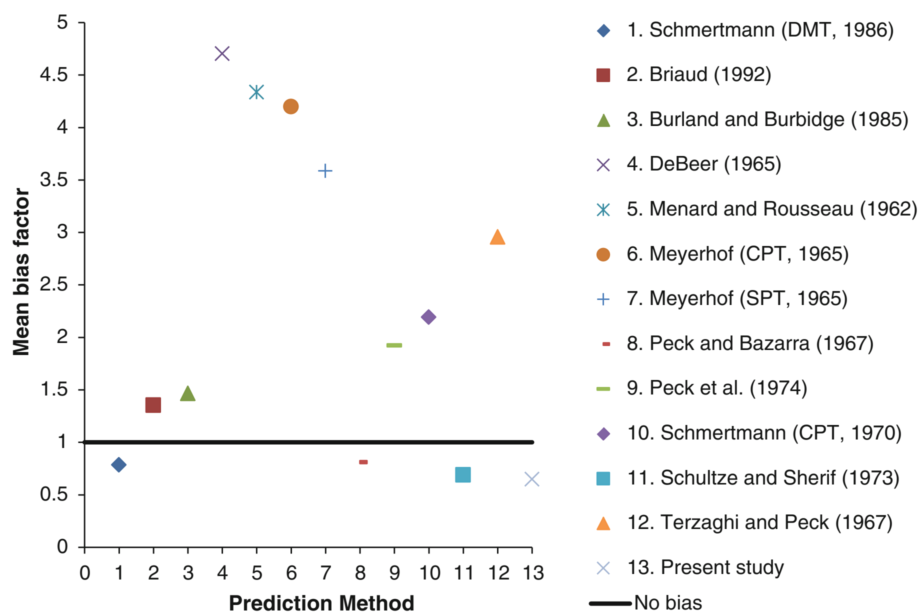

Fig. 10 Variation of mean bias factor of bearing pressure with prediction methods

$$R_n = \frac{\bar{R}}{k_R} \quad (2)$$

$$S_n = \frac{\bar{S}}{k_S} \quad (3)$$

Calibration of these factors is the heart of the Limit State Design, and interested readers can find more details from Becker [2]. The values of Φ and α can be approximated as follows:

$$\Phi = k_R e^{-\theta\beta V_R} \quad (4)$$

$$\alpha = k_S e^{\theta\beta V_S} \quad (5)$$

where k refers to the ratio of mean value to the nominal (characteristic) or specified value of the respective resistance or load variable, V refers to the coefficient of variation of the respective resistance or load variable, β is the target reliability index, which for foundation design, can be taken in the range of 2.5–3.5, which corresponds to a life time probability of failure of foundations on land, θ is the separation coefficient, which is defined as:

$$\theta = \frac{\sqrt{1 + \left(\frac{V_R}{V_S}\right)^2}}{1 + \frac{V_R}{V_S}} \quad (6)$$

Values of V_R and k_R depend on several factors including the site investigation method, quality and quantity of testing, construction quality control, type of foundation, and method of analysis [2]. In practical, the value of V_R/V_S may

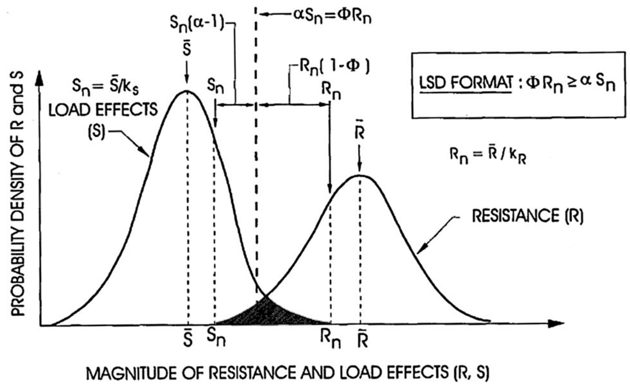

Fig. 11 Limit State Design Philosophy [2]

be in the range of 0.5–5, and the corresponding value of θ varies in the range of 0.7–0.85. The variability of bearing resistance in terms of coefficient of variation, V_R, is well reported by various researchers, and it may vary from as low as 0.08 (obtained from load tests) to as high as 0.5 (obtained from using correlations with SPT). On the other hand, the variability of load component, V_S, varies in the range of 0.07–0.3. The higher values of V_S correspond to live and wind loads, which exhibit greater variability, than dead loads. The values of k_S and k_R vary in the ranges of 0.7–1.0 and 1.1–1.2, respectively. Since the present study deals with load test data, with a mean bearing resistance bias factor of 0.65, higher value of V_R of the order of 0.3 is more justifiable. For k_R of 1.1, V_R of 0.3, θ of 0.75, and β of 3.5, Eq. 4 leads to a resistance factor (Φ) of 0.5. This value of Φ can be directly used in Eq. 1 along with corresponding load factors to design the foundation, which would satisfy the target failure probability of approximately 10^{-4}. To facilitate implementation of LRFD approach in geotechnical engineering, a design example of a shallow foundation is presented in "Appendix: Design Example Based on LRFD Approach" section.

Conclusions

The PLT, which is regarded as the most reliable in situ test to obtain load–settlement response, may yield unrealistic results if not properly conducted. This paper focuses on the analysis of field and laboratory PLT results to help understanding the effects of maintained and non-maintained load during each loading stage on the load–settlement response. Following are the salient conclusions drawn from the study.

(1) From both stress maintained and stress non-maintained laboratory PLTs on remoulded marine clay, it is observed that the load–settlement response for both the cases are quite different. The non-maintained load tests exhibit flatter load–settlement response, and may lead to overestimation of bearing resistance of the soil.

(2) It is also noticed that the soil rebounds if load is not maintained during the load increment, whereas in case of maintained loading the soil does not rebound but settles continuously, with reduced rate of settlement with time.

(3) Equilibrium pressures are attained at lower pressure increments after 15–20 min of load application, in line with the observations of Mohan et al. [10]. However, no such equilibrium pressures are noted at higher pressures on the plate.

(4) The mean bias factor for bearing pressure obtained from the present study is 0.65, which lies between 0.6 and 4.6 reported by Briaud and Gibbens [3]. Based on the results obtained from the present study, a resistance factor of 0.5 is obtained for bearing pressure, which reflects a greater uncertainty in the predicted bearing resistance using non-maintained load test.

Appendix: Design Example Based on LRFD Approach

In this section, the design of a shallow square foundation for an isolated column carrying only dead and live loads is presented based on LRFD approach, and using PLT data. The following hypothetical data is considered to arrive at an appropriate size of foundation.

Nominal dead load carried by the column (S_{n1}) = 500 kN.

Nominal live load carried by the column (S_{n2}) = 500 kN.

Assuming that the bearing capacity of soil is independent of the foundation size, and average (or mean) bearing capacity obtained from PLT = 500 kPa.

Ratio of mean value and nominal value of load (k_S) = 0.7.

Ratio of mean value and nominal value of load (k_R) = 1.1.

Target reliability index (β) = 3.5.

Coefficient of variation of resistance obtained from load tests (V_R) = 0.3.

Coefficient of variation of both dead and live loads (V_S) = 0.2.

Separation coefficient (θ) obtained from Eq. 6 = 0.72.

Resistance factor (Φ) obtained from Eq. 4 = 0.5.

Load factor (α) for both dead and live loads obtained from Eq. 5 = 1.2.

From Eq. 1:

$\Phi R_n = 1.2 \times (500 + 500) = 1,200$ kN.

Nominal bearing resistance of the soil to satisfy Eq. 1 = 1,200/0.5 = 2,400 kN.

Mean value of bearing resistance = $R_n \times k_R$ = 2,400 × 1.1 = 2,640 kN.

Size of the square foundation required based on LRFD approach = 2,640/500 = 2.3 m × 2.3 m.

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