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EXPERIMENTAL STUDIES ON MODEL CAISSONS IN SOFT CLAY UNDER LATERAL CYCLIC LOADING

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ABSTRACT

In this paper, the influence of cyclic load on the deflection of caisson seawall in soft marine clay has been brought out. The load-ground level deflection curves and cyclic degradation of soft clay are presented and discussed under the static and cyclic lateral load for the load eccentricity ratio (e/D) of 1 and caisson embedment ratios (L/D) of 2, 3 and 4 and at the clay consistency index, Ic = 0.42. The ground level deflection variations with number of cycles are reported and discussed. Under cyclic loading with cyclic load ratios (CLR, it is defined as the ratio of applied cyclic load to the ultimate static load) up to 0.6, there is a marginal increase in the lateral capacity of caisson. However, at higher cyclic load ratios, there is a considerable increase in the deflection. From the cyclic load tests, it has been revealed that even up to the cyclic load level of 0.6 with respect to ultimate static lateral load, the deflections are very less and these are comparable to the deflections observed under static conditions corresponding to the same load levels. Under increased load levels of CLR of 0.8 and 0.9, the caisson soil system is approaching failure. Based on the present investigation, from the ground level deflection point of view, it can be suggested that the seawalls can be designed considering the cyclic load ratios even up to 0.6, provided suitable embedment ratios are adopted.

Keywords: Consistency Index, Embedment Depth, Load Eccentricity, Cyclic Load Ratio

INTRODUCTION

Wind and wave loads impose significant lateral load on the foundations of coastal structures and these loads are cyclic in nature. In a few situations, continuous seawalls made of caissons are used as coastal protection works and as breakwaters to protect harbours and these caissons are to be provided with adequate depth of embedment such that there is a good amount of passive earth pressure developed from the embedded portion. In respect of conventional types of caissons and well foundations which are rigid in nature, there are some established methods to estimate the lateral capacities and it should be possible to extend these theories to estimate the stability of the seawall constructed out of the contiguous formation of embedded caissons. There are a few studies reported in literature on the mobilization of ground level deflections in caissons under cyclic lateral loads in clay. Some of the theories related to rigid piles are extended to study the lateral behaviour of caissons. There can be a few limitations in extending the rigid pile concept to predict the lateral behaviour of caissons. Hansen (1961) proposed a method for the estimation of lateral load carrying capacity of piles based on the ultimate lateral resistance concept. This procedure could be used for both cohesive and cohesionless soils. As per this, the earth pressure at the ultimate stage at any depth could be expressed by,

$$q_u = ck_c + qk_q \tag{1}$$

Where, q_u = ultimate soil resistance along the pile length, q = effective overburden pressure, c =

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cohesive strength of clay and k_c and k_q are the factors, which depend on depth and angle of internal friction of soil. Broms (1964) developed a set of charts for estimating the ultimate lateral capacity of rigid piles in both cohesive and cohesionless soils and also suggested a simplified distribution of ultimate soil resistance with depth in terms of passive earth pressure. Matlock (1970) carried out cyclic lateral load tests on an instrumented open-end steel pipe piles of 324 mm in diameter driven into slightly over consolidated marine clay deposit. The soil resistance was found to decrease with number of cycles and after a few cycles of loading, it was stabilised. However at deflections equal to values exceeding 20% of the pile diameter, there was a progressive deterioration in the resistance under continued cyclic loading.

Franke (1973) carried out lateral load tests on an instrumented bored pile of 1.3m diameter and 16.5m embedded length in a clayey soil. The pile was subjected to both one way and two way cyclic lateral loads. The results indicated that two way cyclic loading had the same effect as that of one way cyclic loading. Bushan et al. (1979) conducted large-scale field tests on laterally loaded drilled shafts in stiff clays and presented variations in lateral deflections and bending moments with depth in drilled shafts and suggested that the existing procedures for the analysis of laterally loaded piers in stiff clay generally appear to provide conservative predictions of the lateral load-deflection relationships for the rigid piers in stiff clays. Price and Wardle (1979) carried out static and cyclic lateral load tests on vertical steel tubular piles of 5.1 m length and 168mm diameter jacked into London clay. The results showed that the coefficient of subgrade reaction from cyclic loading was higher than the value estimated from static load test. Cyclic loading caused a reduction in the magnitude of the moments induced in adjacent horizontally loaded piles and this indicated an increase in the shear strain in the soil between the loaded pile and nearby unloaded piles due to a reduction in the shear modulus of that soil. Poulos (1982) presented an analysis by extending the elastic continuum analysis for the deflection of a pile in clay subjected to static and cyclic lateral loading. In this analysis, a number of theoretical solutions are presented to illustrate the factors influencing the increase in deflection and bending moment in the pile with an increase in number of cycles and cyclic load level. The results obtained based on this analysis are further compared with the observed behaviour of single pile in the field and there were found to be satisfactory comparisons.

Dunnavant and O' Neill (1989) presented a series of full scale cyclic lateral load tests on instrumented piles of varying diameter at a test site in submerged over consolidated clay. Improved p-y criterion for submerged stiff clay was developed. Test data indicated that appreciable cyclic degradation did not begin until the pile head displacement had reached to a value of about 1.01 B (B is the diameter of pile) but once degradation started, it did not appear to stabilise within 200 cycles. The main source of degradation was due to the development of permanent gap around the piles intensified by hydraulic erosion. Two simple methods for determining the effect of lateral cyclic loads on a pile in sand were presented by Long and Vanneste (1994). These were essentially based on the analysis carried out using published results of 34 full-scale cyclic lateral load tests. One method employed a closed form solution using a beam on elastic foundation analysis with a linearly increasing soil reaction modulus and there was degradation in the modulus under cyclic loading. In the second method, the resistance obtained by static p-y curve was deteriorated to account for the effects of cyclic lateral load. The most important parameters found to govern the behaviour of piles during cyclic loading were the characteristics of the cyclic load, method of installation and soil density. Mayne et al. (1995) investigated the behaviour of free-head rigid drilled shafts under static and cyclic lateral loading using laboratory models tested in relatively large test chambers. The results of the lateral load and moment tests indicated a high degree of nonlinearity in the monotonic static load-displacement response, but it could be represented adequately by a hyperbola. This hyperbolic formulation also provides a reference backbone curve for the cyclic loading behaviour. Brettmann and Duncan (1996) presented equations for the non-linear relationships used in characteristic load method (CLM) of analysis of laterally loaded piles and drilled shafts to estimate accurately ground-line deflections and maximum bending moments for free-head and fixed-head piles in both clay and sand. Zen et al. (1998) reported the field application of caissons and suggested the use of suction force in the installation of caissons. It was stated that the penetration of caisson enhances the horizontal resistance of the foundations.

Considering the soil as normally consolidated clay in which undrained shear strength increased with depth, a soil flow mechanism was developed by Yuxia and Randolph (2002) for caissons with embedment depth ratios up to 5. It was found that with increasing embedment depth ratio, the soil flow mechanism changed from surface failure to a deep cavity expansion mode and this transition occurred at higher embedment depth ratio, L/D. From the aforementioned review, it is felt that there is a need to make a further study in predicting the lateral behaviour of caisson subjected to cyclic loading in clayey soil. An attempt has been made to arrive at predictions for lateral capacity of caisson in marine clay using the test results obtained from a controlled testing in the laboratory.

EXPERIMENTAL WORK

Soil Used

Marine clay from the coastal deposit of Chennai, located in the east coast of the India was used in this investigation. The index properties of marine clay tested are Liquid limit, LL = 48% and Plastic limit, PL = 18%. These values were obtained from standard tests carried out as per the relevant ASTM Standard [ASTM- D 4318-2000]. The soil beds in the test tank were formed at a placement moisture content conforming to consistency index, Ic = [(Liquid limit -Placement moisture content)/Plasticity Index] value of 0.42. This Ic value adopted in this investigation suits the range of value in the field for this particular deposit. The undrained cohesion of clay formed at this Ic is 12.2 kPa. In all the tests, the clayey soil was mixed with enough water to get the desired consistency and cured for a period of at least 2 days.

Model Caissons Used

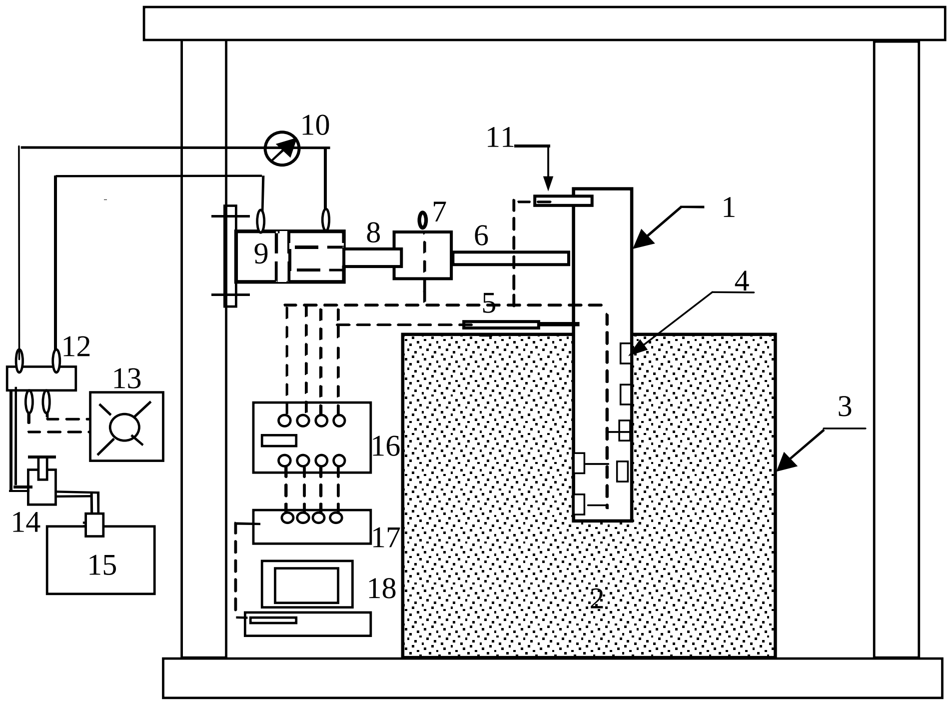
A model caisson made out of mild steel pipe of 105 mm outer diameter with 10 mm wall thickness was used. The embedment length (L) to diameter (D) ratios (L/D) of model caissons investigated were 2, 3 and 4. The load was applied at a value of load eccentricity, (e) and with an eccentricity ratio, e/D = 1.0. The rigidity of the model caissons has been estimated as per the guidelines suggested by Poulos (1971) and it is confirmed that for all the placement soil conditions and the caisson material, the model caissons used are rigid. The flexibility factor as per the Poulos (1971) is expressed as:

$$k_R = \left(\frac{E_p I_p}{E_s L_e^4} \right) \tag{2}$$

Where, k_R = Flexibility factor, E_p= Caisson material's Young's modulus (kPa), I_p= Moment of inertia of caisson material (m^4), E_s = Soil modulus is taken as equal to 40 c_u, [as per the Poulos, 1971], c_u = undrained shear strength (kPa) and L_e = Embedment length of caisson in soil (m). As per this criterion, if k_R is greater than or equal to 0.01, the caisson or pile is considered to be rigid. (Meyerhof, 1976). From the calculations made, it is observed that the k_R is more than 0.01 for all the tests. Hence, all the model caissons used in this investigation are rigid.

Test Setup

The tests were conducted in a rectangular test tank of size, 1200mmx 800mm x 900mm. The test tank was chosen such that the size effects of model caissons adopted in this study were considered to be minimum. As per the analysis suggested by Yegian and Wright (1973) using the finite element approach, it has been confirmed that the size effects are negligible, if the radial distance of the test tank in the direction of loading is more than 6 to 8 diameters of the model. In this setup adopted, the radial clear distance between the outer face of caisson model and the edge of test tank in the direction of lateral loading is more than 6D. In this experimental setup, the lateral load was applied through an extension rod placed at certain height above the soil bed and loading was gradually applied in stress controlled load increments. Considering all the aspects of field type of loading, a special stress controlled pneumatic loading device was designed and fabricated to apply lateral loading. The load cell coupled on to the piston rod measured the lateral load acting on the model caisson. The inductive type LVDT (Linear Variable Differential Transducer) was placed at the ground level to monitor the ground level deflections. All the observations were made through the Controlled Data Acquisition System (CDAS), which was linked to the personal computer to provide all critical controls, timing and data acquisition functions for all the measuring devices used. The measurements were continuously logged into the computer through data acquisition system. The schematic arrangement of the experimental setup is shown in Figure 1.



1. Model Caisson, 2. Soil, 3. Test tank, 4. Pressure cell, 5. LVDT, 6. Extension Rod, 7. Load cell, 8.Piston Rod, 9.Pneumatic power cylinder, 10. Pressure gauge, 11. Tilt meter, 12. Solenoid valve, 13.Electronic Timer, 14. Pressure Regulator, 15. Compressor air chamber, 16. Carrier frequency amplifier, 17. BNC box, 18. Data acquisition system.

Figure 1. Schematic diagram of test setup

Method of Filling up of Tank and Installation of Model Caissons

Clayey soil was mixed thoroughly with the required amount of water to maintain the desired consistency index, Ic of 0.42 and was cured for 2 days to ensure uniformity in the moisture content. After placing the model caisson in vertical position in test tank, soil was placed in layers of 50mm thickness with hand packing and pressed by jacking a template to remove entrapped air and to ensure homogeneous packing. As soil beds were formed at soft consistency, there was no difficulty in forming fairly homogeneous strata. The uniformity in density was checked by measuring the density at various depths and radial distances. The near full saturation of soil was confirmed by the measurements of pore water pressure parameter of B = 0.98 to 0.99 (Bishop & Henkel, 1962) from the samples taken and tested in triaxial test assembly.

Static and Cyclic Lateral Load Test

The static and cyclic lateral loads on the caisson were applied using pneumatic loading system. The loading consisted of pneumatic power cylinder, electronic timer and solenoid valve arrangement. These details are shown schematically in Figure1. The ultimate load was evaluated from the load-ground level deflection plots corresponding to a ground level deflection equal to 0.2 times the diameter of model caisson as per the criterion suggested by Broms (1964). The ultimate or failure load could be reached by gradually increasing the load in load increments of 10 to 12 in number and at each of the load increment; it was waited till the rate of ground level deflection approached a negligible value. The cyclic load is applied in the form of Cyclic Load Ratio (CLR).

RESULTS AND DISCUSSIN

In this study, a number of lateral load tests under cyclic loading are conducted on rigid embedded model caissons into soft clay. The cyclic load levels are varied from 50 to 90% of the ultimate static lateral load. The cyclic loading is continued until a stabilised value of deflection has reached and it is presumed that the deflections are found to be stabilized fairly constant over 300 successive load cycles. The results pertaining to the mobilisation in ground level deflection and degradation in the cyclic stiffness under cyclic loading are presented and discussed in the following section.

Static Lateral Load-Ground Level Deflection Curves

The safety of a caisson used in the seawall construction is controlled by the settlements and deflections. The variation in ground level deflection under static loads is presented in Figure 2 for different embedment depth ratios of caisson corresponding to Ic=0.42 and e/D=1. Cyclic load levels are fixed based on the ultimate static lateral loads obtained from these curves. The ultimate static lateral capacities are obtained as per the Broms (1964) criteria. As per this criterion, the load corresponding to the ground level deflection equal to 0.2D (D= diameter of caisson) from the lateral load-ground level deflection curves is taken as ultimate static lateral load. The ultimate static lateral loads observed for L/D = 2, 3 and 4 are 0.24kN, 0.44kN and 0.62kN respectively at e/D=1 and Ic=0.42. From the above static loads, the lateral loads corresponding to cyclic load ratios of 0.5, 0.6, 0.7, 0.8 and 0.9 are 0.12kN, 0.144kN, 0.168kN, 0.192kN and 0.216kN respectively for L/D=2; 0.22kN, 0.264kN, 0.308kN, 0.352kN and 0.396kN for L/D=3 and 0.31kN, 0.372kN, 0.434kN, 0.496kN and 0.558kN respectively for L/D=4.

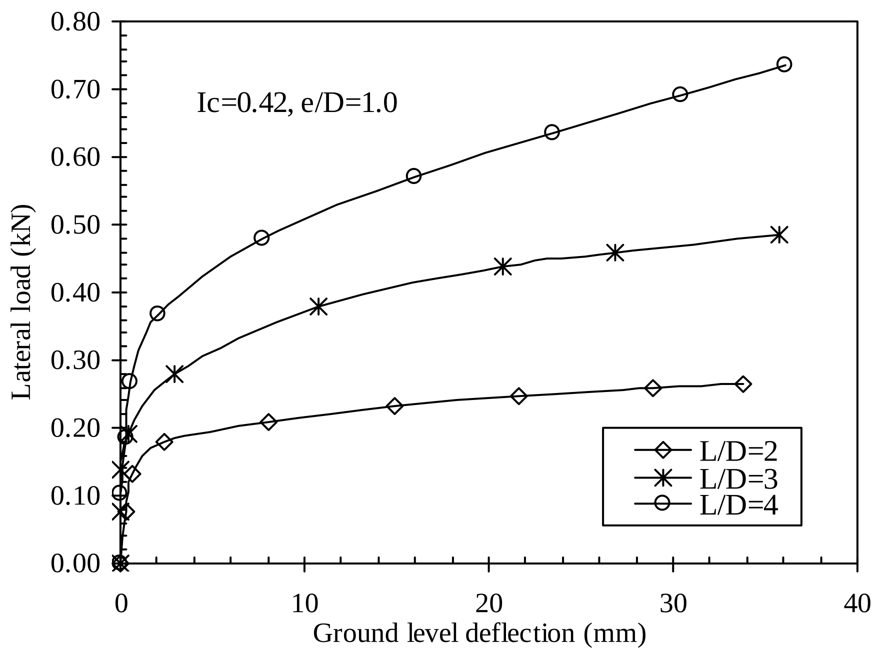


Figure 2. Variation in ground level deflection under static lateral loading

Ground Level Deflections of Caissons under Cyclic Lateral Load

Cyclic lateral load tests are conducted on caisson models embedded in clay bed formed at a consistency index, Ic of 0.42 and these loads are applied at load eccentricity ratios, e/D of 1.0 for caisson embedment depth ratio, L/D of 2, 3 and 4. The ground level deflections are measured at different number of cycles for the cyclic load ratios, CLR of 0.5, 0.6, 0.7, 0.8 and 0.9. The variations in the ground level deflection with number of cycles at different cyclic load ratios are presented in Figures 3 to 5. These are the results obtained from the tests carried out on caisson models with embedment ratios (L/D) of 2, 3 and 4 and with load eccentricity ratio (e/D) of 1.

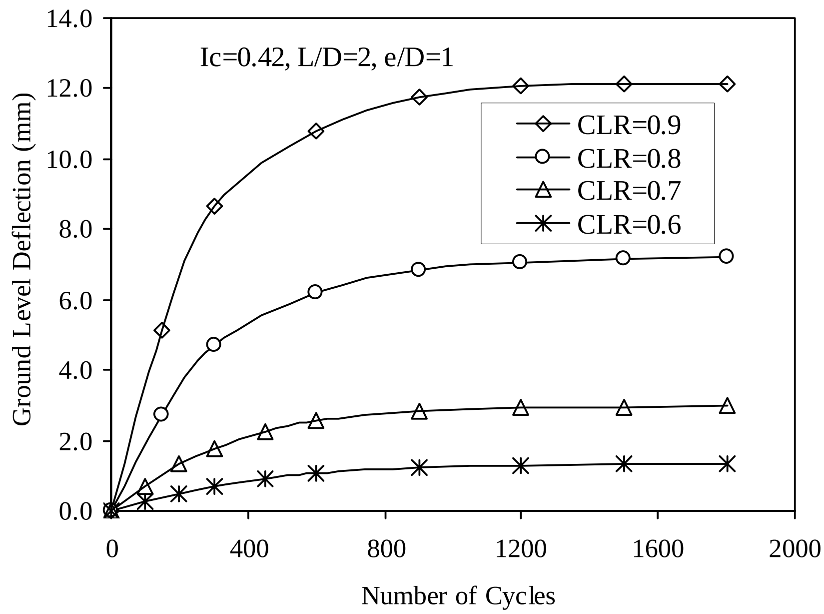


Figure 3. Variation in ground level deflection with number of cycles for different CLR values

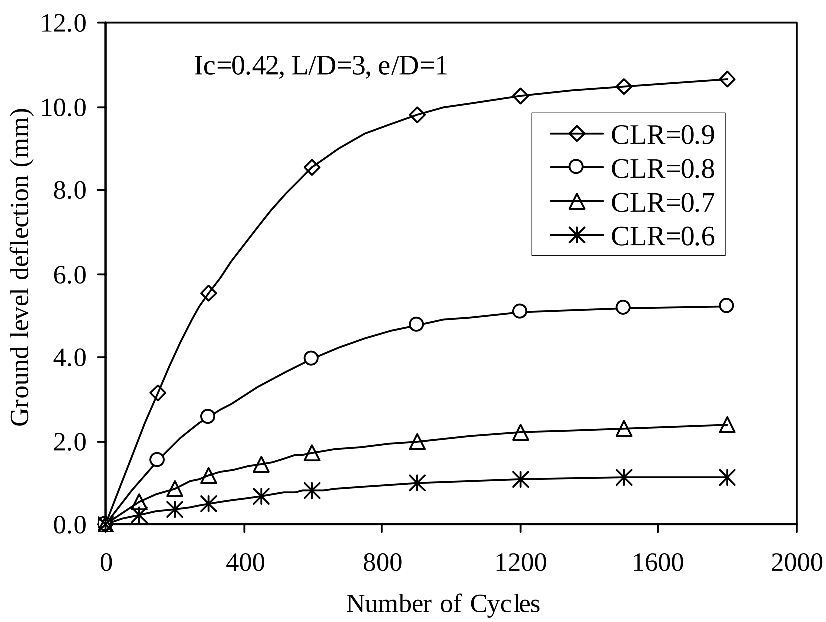


Figure 4. Variation in ground level deflection with number of cycles for different CLR values

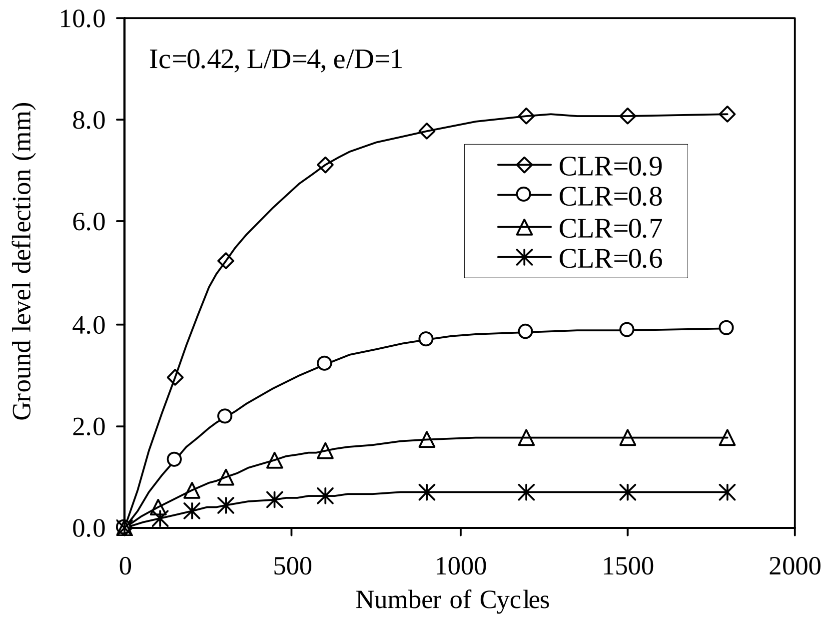


Figure 5. Variation in ground level deflection with number of cycles for different CLR values

From these results, it can be seen that as the number of cycles increases, ground level deflections increase up to certain number of cycles and thereafter, they get stabilised. In the initial stage, there is almost linear increase in deflection with number of cycles and beyond this range, there is a curvilinear pattern of variation. With further increase in load cycles, there is not much increase in the deflections and the curve between deflections and number of cycles is asymptotic. In all these cases, this increase in trend in deflection is seen up to about 900 cycles and beyond which, the deflection is almost constant. It can be observed that the cyclic load level has significant influence on the deflections. As the cyclic load ratio increases from 0.6 to 0.9, the ground level deflections are on the increase and between CLR levels of 0.6 and 0.9, there is approximately 10-fold increase in the deflection level. The mobilisation in the ground level deflection is observed up to 1800 number of cycles. From these results, further it can be seen that as the embedment depth of caisson increases, from L/D = 2 to L/D = 4, there is a reduction in ground level deflection and this reduction is by about 30%, for the same cyclic load ratios and for the same range of cycles. This typical behaviour of increase in deflection at lower embedment depth ratios is due to less mobilisation in the passive resistance of clay.

The approximate values of the ground level deflections mobilized at 1800 number of cycles for L/D = 2, 3 and 4 and at e/D = 1 and Ic = 0.42 are tabulated in Table 1. From the results presented in this table, some interesting trends can be observed in the ground level deflections. With increase in cyclic load ratio, there is a significant increase in the ground level deflection.

TABLE 1. Mobilised ground level deflection under cyclic lateral loads at 1800 cycles

CLR	e/D=1		
	L/D=2	L/D=3	L/D=4
0.6	1.3	1.1	0.75
0.7	3.0	2.45	1.75
0.8	7.0	5.2	3.95
0.9	12.2	10.8	8.05

Further, it is observed that at lower embedment depths of caissons, the stabilised ground level deflections are found to be higher as compared to the deflections at higher embedment depth ratios and these are explained in terms of passive earth pressures mobilised. Further, it can be summarised that as L/D is reduced from 4 to 2, the percentage increase in the ground level deflection at 1800 cycles corresponding to cyclic load applied at e/D = 1 and at an Ic = 0.42 are 73%, 71%, 79% and 51% at cyclic load ratios (CLR) of 0.6, 0.7, 0.8 and 0.9 respectively. The changes in the ground level deflections can be further summarised with the CLR. From the tests carried out with e/D=1 and at stabilized deflections at 1800 cycles, it can be seen that as the cyclic load ratio increases from 0.6 to 0.9 ground level deflection increase by 8.38 times the deflections corresponding to CLR of 0.6 and L/D=2. Similarly at L/D=3, the deflections increase by 8.8 times the deflections corresponding to CLR of 0.6. For the tests with L/D=4, the increase in deflections are 9.73 times the deflections at CLR=0.6. Further, to understand the behaviour of ground level deflection under cyclic load, the plots are drawn for different number of cycles as shown in Figures 6, 7 and 8.

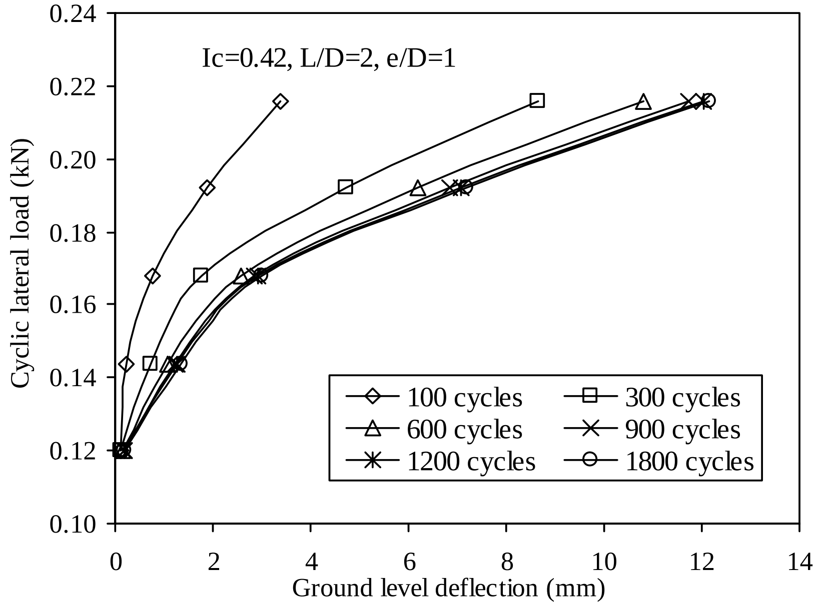


Figure 6. Variation in ground level deflection under cyclic lateral load for different cycles

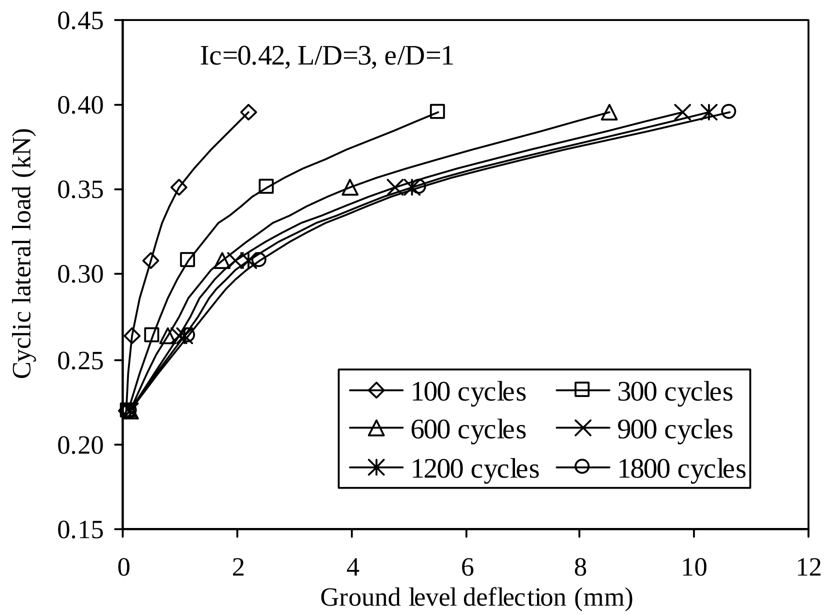


Figure 7. Variation in ground level deflection under cyclic lateral load for different cycles

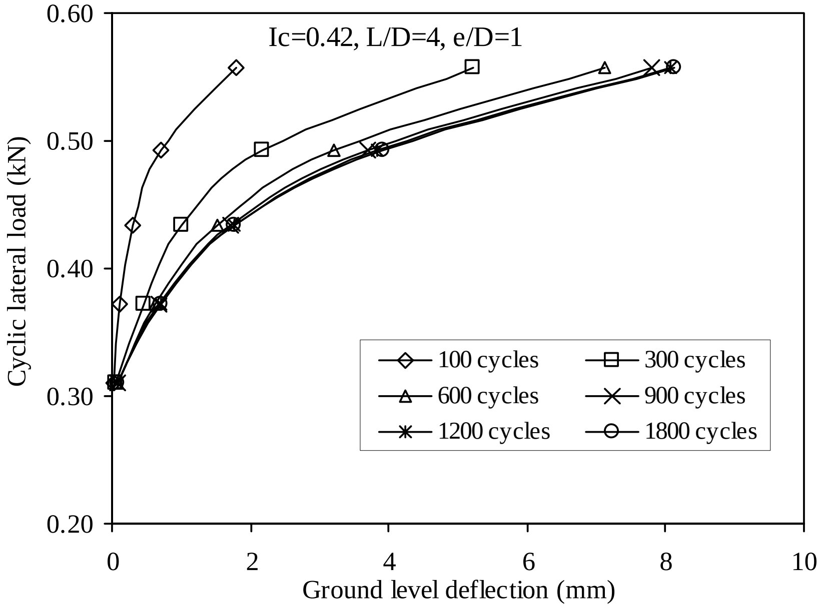


Figure 8. Variation in ground level deflection under cyclic lateral load at different cycles

In all the figures, typically it can be seen that the deflections mobilized are observed to be more at higher number of cycles. It can be noticed that there is a marked increase in the ground level deflections only in the range of cycles up to 900 and beyond these cycles, the deflection is almost remaining the same. These plots suggest that the curves are getting merged beyond 900 cycles of loading.

The Degradation in the Cyclic Stiffness

The safety of the seawall formed out of contiguous embedded caissons is controlled by the soil strength and the lateral deflections. Stiffness parameter representing the soil is used in many of the theoretical estimates of the lateral deflections. Due to the possible damages under cyclic loading, there can be considerable reduction in the stiffness of the soil and this aspect has been examined in this section. The cyclic stiffness of the caisson soil system is defined as the ratio of the cyclic load applied to the cyclic deflection at the ground level corresponding to a number of cycles, N. The variation in cyclic stiffness with the number of cycles is brought out in Figures 9 to 11. These values are obtained from tests conducted on caissons embedded in soil bed with Ic = 0.42 for L/D = 2, 3 and 4 and for the conditions of e/D = 1. These bring out that at any number of cycles, the cyclic stiffness decreases with increase in CLR.

At low CLR values, there is not much change in this value with number of cycles. However, at high CLR values of 0.8 and 0.9, cyclic stiffness undergoes a steady degradation with number of cycles and finally leads to failure of the caisson. From these results, it can be inferred that for the cases of L/D = 2, 3 and 4 and e/D = 1, in general, it can be stated that there is significant reduction in modulus with number of load cycles in the range of 400 to 600 cycles and beyond which these values are constant and this is in the same pattern as that of deflection. From Figure 9 representing L/D = 2 and e/D=1 at Ic = 0.42, for lower values of CLR = 0.5 and 0.6, the stabilised values of cyclic stiffness or modulus values are about 700 N/mm and 150 N/mm respectively. The initial stiffness value is obtained from the load-ground level deflection curve (Figure 2) for the respective condition under static lateral loading case and for the similar conditions of Ic = 0.42, L/D = 2 and e/D = 1 and this stiffness value is about 400 N/mm. From the above values, the initial stiffness value obtained from static test is lower than the stabilised value of cyclic stiffness at CLR = 0.5. This indicates that the load applied at CLR = 0.5 is not causing any movement in the caisson and in turn there is no damage in the soil system.

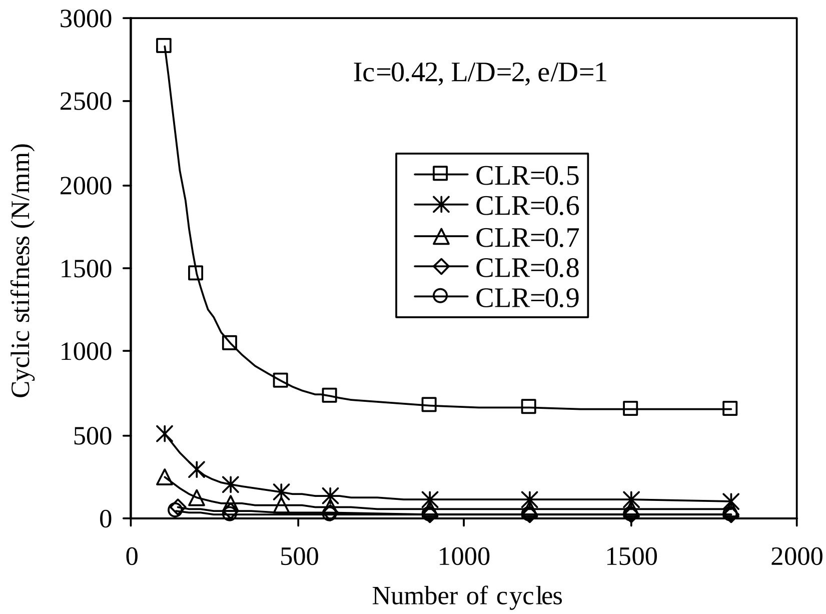


Figure 9. Variation in cyclic stiffness with number of cycles for different CLR values

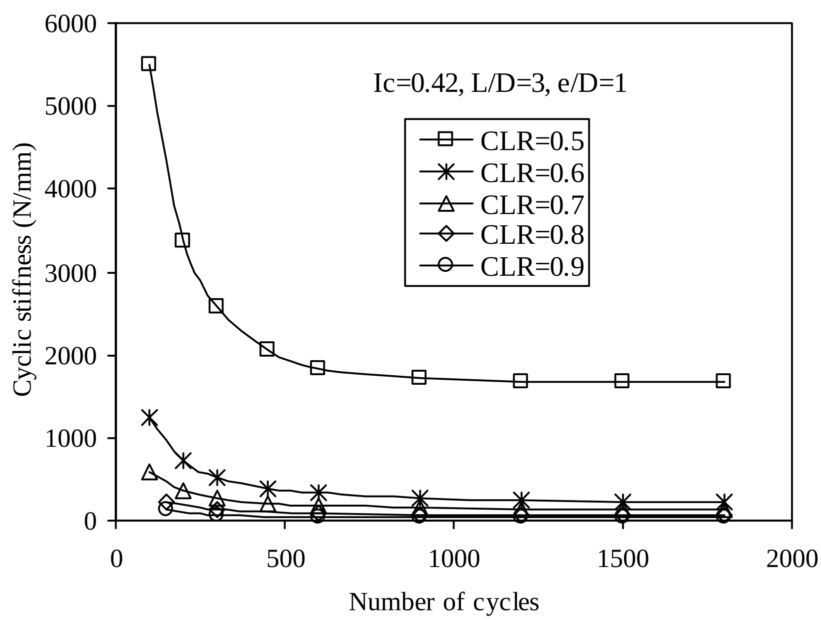


Figure 10. Variation in cyclic stiffness with number of cycles for different CLR values

It can be seen that the cyclic loading has brought down these values significantly. At CLR of 0.7, this stiffness value has come down to a value of 70 N/mm at the stabilised state. Corresponding to CLR of 0.8 and 0.9, the values are so small and for all practical purposes, it can be assumed that the caissons have suffered with an extensive tilting. However with increase in L/D ratio to 3 and 4 corresponding to same value of e/D = 1, it could be seen that the stiffness values at the stabilised condition are considerably higher. For example, for L/D=3 (Figure 10) for CLR of 0.5, the stabilised stiffness value is 1750 N/mm. For L/D of 4 (Figure 11) this stabilised value is nearly 4000 N/mm. For L/D = 4 with CLR of 0.7 the stabilised value of stiffness is about 650 N/mm. From this, it could be clearly seen that for L/D of 2, for all design purposes, the designed stress should be arrived in such a way that it should correspond to CLR of 0.5. However with increase in L/D, it can be designed for CLR of even up to 0.7. In comparison with other options for the construction of seawall, the caisson-supported seawall may prove to be economical. All these results suggest that increase in deflection during cyclic loading is of main concern rather than the absolute lateral capacity. These suggest that with increase in L/D, the system is better. Considering the scour depths, it is possible to arrive at suitable L/D value. However, from the construction point of view, it may not be possible to adopt L/D values beyond 4.

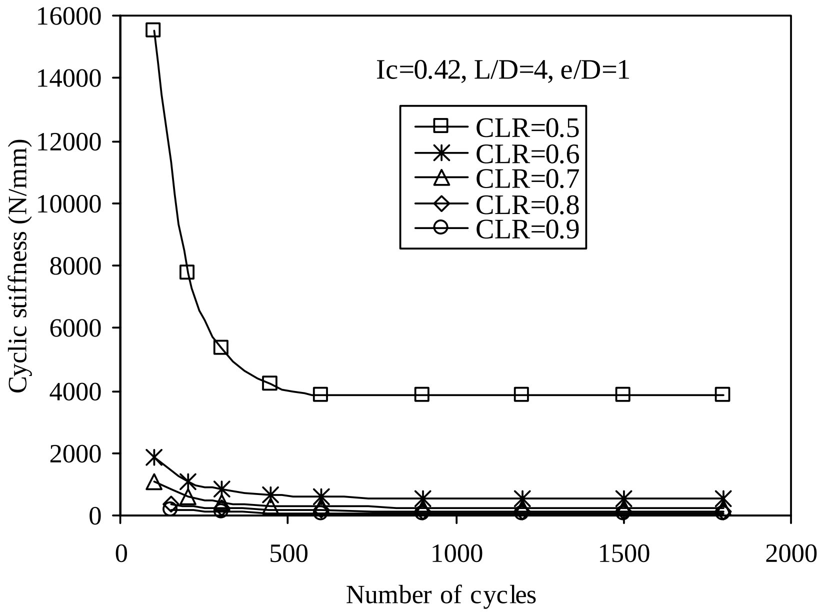


Figure 11. Variation in cyclic stiffness with number of cycles for different CLR values

CONCLUSIONS

From the cyclic load tests, it has been revealed that even up to the cyclic load level of 0.6, the deflections are very less and these are comparable to the deflections observed under static conditions corresponding to the same load levels. Under increased load levels of CLR of 0.8 and 0.9 system is approaching failure. Within the first 800 to 900 cycles there appears to be an increase in the deflection and beyond these cycles there is stabilization in the ground level deflection. The safety of the system can assumed to be interpreted in terms of cyclic deformations, which can be predicted based on the cyclic stiffness value. The safety of the caisson at higher levels of cyclic loads can be improved by increasing the embedment depth ratio, L/D. The results suggest that with increase in L/D, the system is better. Considering the scour depths, it is possible to arrive at suitable L/D value. However, from the construction point of view, it may not be possible to adopt L/D values beyond 4.

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