Field loading tests on instrumented piles – Hazira Slug Catcher project
Essais de chargement en place sur des pieux instrumentés – Le projet 'Hazira Slug Catcher'

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SYNOPSIS: Economic design of Pile foundation requires realistic basic design parameters of adhesion and coefficient of lateral earth pressure. These parameters have been determined by analysing the load-movement behaviour of Piles based on the data collected from the full scale loading tests conducted on the Instrumented Piles installed at the Hazira Gas Complex Site. The validity of these parameters has been established by the excellent agreement that has been shown to exist between the computed and the actual load settlement curves obtained from other field loading tests.

1 INTRODUCTION

The gas produced offshore at the South Bassein field in the vicinity of Bombay High is transported to Hazira by offshore and onshore pipelines. A slug catcher has been installed at this site to separate the condensate from the gas by gravity flow and also to store its total hold up during pigging operations of the pipeline. The slug catcher is a piped structure consisting of 48 fingers, each being 1.22 m diameter and 478 m long. Due to criticality of slopes of the separator section and the storage section minimal differential settlements were specified. Lack of availability of suitable bearing capacity of subsoils coupled with stringent settlement requirements discounted use of shallow foundations in favour of piled foundations.

Future development plans of the Gas Complex envisaged supporting all major facilities on piles. Appreciating the economy that could result on account of the enormity of piles, it was indeed essential to optimize the pile design. With this in view, two driven cast-in-situ piles of 450 mm nominal diameter identical in all respects with the working piles were installed. These piles were constructed by driving a casing closed at the bottom by a conical shoe. After the casing was driven to the required depth in accordance with the field driving criteria, the reinforcement cage carrying the necessary instruments was lowered. The pile was then concreted by lifting the casing in stages. After expiry of the curing period of 28 days, the piles were load tested to failure by applying direct incremental loads. During the tests, the data on butt settlements as well as that from the strain gauges were recorded. The paper analyses this data to obtain load transfer characteristics of the piles and also establishes design factors required for an economical and reliable design of pile foundations at the Hazira Site.

2 SITE CONDITIONS

The subsoil conditions at the test pile locations separated by a distance of nearly 450 M are shown in figure 1. Below a site grading fill of 0.8 to 1.4 m thickness, the subsoils

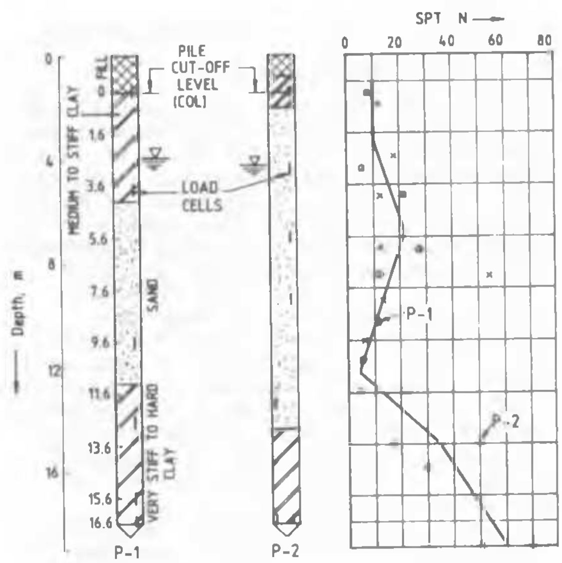

Figure-1. Subsoil Profile at the Hazira Site

consisted of dark brown clay (stratum I) followed by black sand (stratum II) which, in turn, was underlain by yellow/brown clay (stratum III) A comparison of the natural water contents and Atterberg limits indicated that the soils in stratum I and III comprised of medium to stiff and very stiff to hard overconsolidated clays, respectively. The intermediate sand stratum II was, in general, in medium dense state.

The undrained shear strength of the clays was determined by two methods. In the first method, it was obtained from the triaxial unconsolidated undrained (TUU) tests on undisturbed samples collected from clays of strata I & III and in the second method, an empirical relationship given by Terzahi and Peck (1967) was used. This relationship is represented by $S(kg/cm^2)=N/15$ where N designated the recorded SPT N. Figure 2 demonstrates that while the undrained shear strengths determined by these methods showed good agreement in case of medium to stiff clays

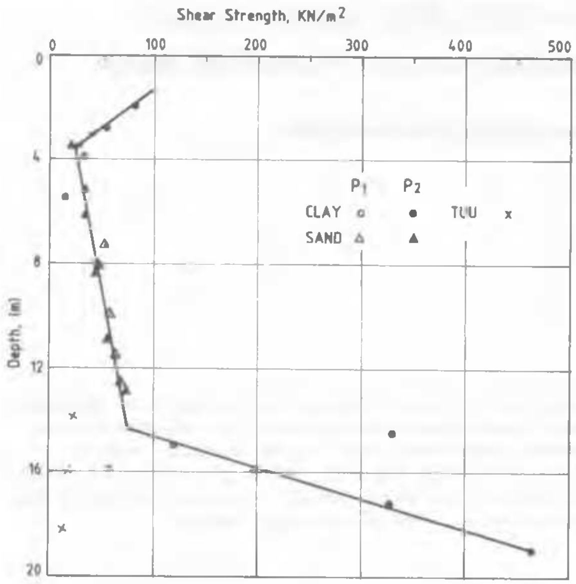

Figure-2. Shear Strength Profile

(N ranged from 8 to 12), the comparison was poor for clays having very stiff to hard consistency. This is attributed to the difficulties of obtaining good undisturbed samples from the very stiff clays. That in such clays the Terzahi and Peck relationship provides a reasonable assessment of the undrained strength has been amply demonstrated by several previous investigators (Reese et.al. 1973 & 1976). The undrained shearing strength of clays based on this empirical relationship plotted as solid line in figure 2 was adopted for analyses presented in this study.

The shear strength of sands prior to installation of the piles was determined by using S=P'. tan Ø' where P'= effective overburden pressure and Ø'= effective friction angle as estimated from the observed N values using figure 1 of the Indian Standard 6403-1981.

A review of the physical properties of the subsoils revealed them to be fairly uniform. Accordingly, analyses of pile P1 only which were typical of the area, are presented and discussed herein.

3 PILE INSTRUMENTATION

The test pile was instrumented with strain gauge type load cells (ref.figure 1) which consisted of an outer protective steel casing and an inner steel tube to which four electrical resistance strain gauges were fixed to form a bridge. The annular space between the casing and the tube was filled with silicon rubber to make the cell water proof. The system was fully temperature compensated. The stress concentration during loading was avoided by making stiffness of the load cell of the same order as that of concrete. The assembly was fitted to one of the reinforcement bars by welding through a small mild steel flat. The cables from the various load cells were connected to the read out unit placed on the ground surface. The continuity checks were made which revealed that nearly all the gauges worked satisfactorily. The top level load cell was used as calibration to convert the measured

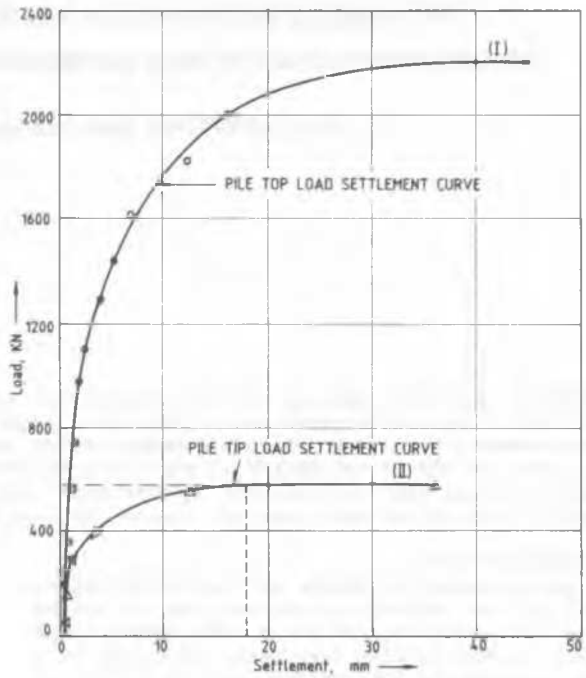

Figure-3. Load-Settlement curves of Pile

strains into respective loads at various levels down the shaft.

4 ANALYSIS OF TEST RESULTS

The load-butt settlement as well as the tip load-tip settlement relations that were obtained are shown in figure 3. The load distribution Vs depth curves derived from the load cell data are plotted in figure 4. Load transfer Vs pile movement curves along various shaft lengths are illustrated in figure 5. This figure was developed by using the experimental data on load distribution, butt settlement and elastic properties of the pile. The data from this figure was, in turn, used to predict the static capacity of the pile based on the method suggested by Coyle and Reese (1966). A comparison of the theoretically computed load-settlement curve with the actual load settlement curves for some field loading tests conducted elsewhere at the site is shown in figure 6.

5 DISCUSSION OF RESULTS

The load-butt settlement relationship presented in curve I (figure 3) is typical of a predominantly friction pile. This relationship characterised by the existence of a peak followed by almost a straight line indicating large increase in butt settlement without increase in load at the pile top can be attributed to lack of high tip resistance. The ultimate load reached is about 218 tons and the ultimate tip load is 58 tons (curve II).Balance of the 160 T load, which is 73.4 percent of the ultimate load, is supported by friction along the pile shaft. At the design load of 76 tons, however, the base supported a mere 16 ton load (21 percent of the design load) while the contribution by friction is quite significant

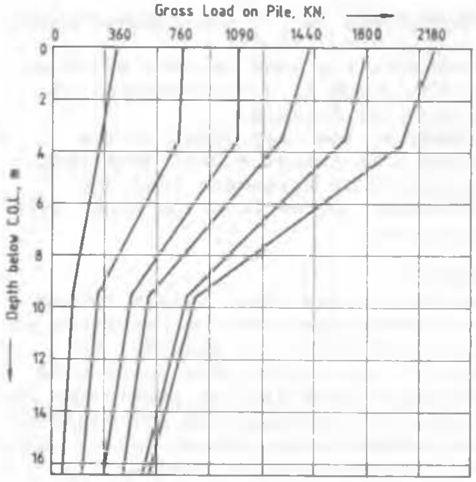

Figure-4. Load Distribution Curves

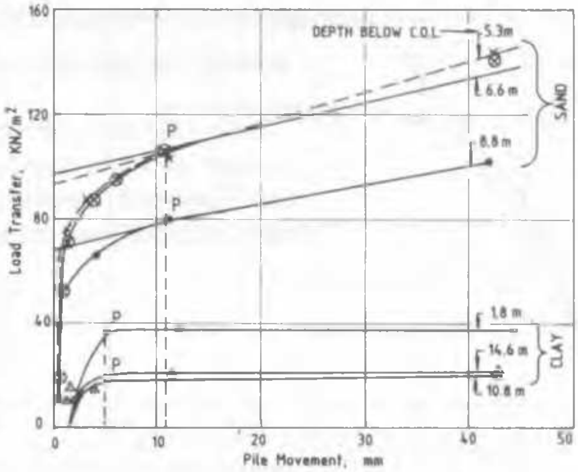

Figure-5. Load Transfer vs Pile Movement Curves

(79 percent, refer figures 1 & 4).

The critical point movement corresponding to to the ultimate point bearing of 58 ton (365 t/m²) is approximately 18 mm which is 4 percent of the pile diameter. This critical movement agrees fairly well with other investigators (Vijayvergiya,1977,Table 2) who indicated the tip movement at failure in clays to range from 4 to 6 percent of the pile diameter.

5.1 Load-Transfer Studies

Figure 5 demonstrates that the load transfer is a function of the pile depth, its movement as well as the type of soil in contact with the pile surface. Small pile movements in the initial stages result in a high rate of load transfer and this rate decreases as more and more load is transferred to soil till a peak as indicated by point P is reached (figure 5). Beyond this point, however, the pile movements generally increase disproportionately to the increase in load transfer. The peak load transfer in clays is seen to range from about 20 KN/m² to 36 KN/m² depending upon their consistency. It should be noted that this peak transfer is higher at shallow depth of 1.8 m than that observed for larger depths e.g.10.8 m and 14.6 m. This observation is in consonance with the nature of the clay which being of medium consistency at shallow depth probably moves inward to make contact with the pile subsequent to withdrawal of the casing in contrast to the very stiff to hard clays at large depths. As is expected,the peak load transfer in sands is considerably higher (80 to 105 KN/m²) than in clays. The critical movement corresponding to the peak load transfer in clays is observed to be 5 mm only irrespective of the consistency of the clay while in sands,it is about 11 mm. Reese et.al.(1973) also observed critical movements of the same order in their investigations.

The load transfer factor (or adhesion factor) α, representing the ratio of peak load transfer to the shear strength of clay obtained from figure 2 is summarised in table 1. The analysis performed revealed that the load transfer factor in medium to stiff clays varied from 0.4 to 0.65 with an average of 0.5. It had a low value of 0.17 for very stiff to hard clays. These findings are consistent with the general observation that the load transfer decreases with increase in shear strength of the clays.

Table 1 : Design Factors from Instrumented Field Loading Tests

S. No.	Depth of Pile below Cut-off level (M)	Soil Type	Design Factor α	Design Factor K
1	1.8	Medium to stiff clay	0.65 } Av.= 0.5	-
2	10.8	Medium to stiff clay	0.40 }	-
3	14.8	Very stiff to hard clay	0.17	-
4	5.3(N=22)	Compact sand	-	2.03 } Av.= 1.9
5	6.6(N=17)	Compact sand	-	1.84 }
6	8.8(N=11)	Near Compact sand	-	1.2

The peak load transfer in sands is obtained from the product K.P'.tanδ where K=coefficient of lateral earth pressure, P'= effective over-burden pressure at the relevant depth and δ = the angle of wall friction taken as ϕ in accordance with the Indian Standard 2911-1979; ϕ having been determined from the recorded N as per the Indian Standard 6403-1981. The latter is based on the fact that no significant change in penetration resistance would occur following driving of a single pile and that any tendency to cause increase in N was partly offset by removal of the soil overburden above the pile cut off level. The peak load transfer derived from this study yielded an average K=1.9 for compact sands and 1.2 for near compact sands. These values lie within K=1 to 3 as specified in IS 2911-1979. However, the derived K is higher than 1 as proposed by Reese (1964). K depends upon the degree of packing against the pile wall

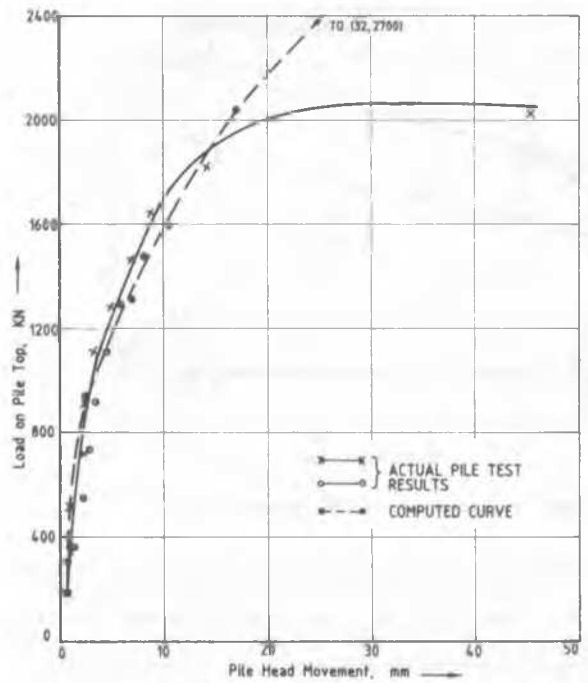

Figure-6. Actual and Computed Load Settlement Curves

which is influenced by the displacement of soil as well as the vibrations induced in soil during driving. It appears that the process of arching which prevents sand to potentially act against the pile surface so as to limit the K factor to 1, is not entirely borne out by the present investigation. However, more data on the Hazira soils are required to confirm this phenomenon.

The agreement between the computed load settlement curve with those actually observed during the full scale loading tests on other piles was found to be highly satisfactory(see figure 6) except the stage of incipient failure wherein the erratic behaviour of the pile due to its highly disturbed load distribution pattern does not permit a meaningful analysis of the results. Generally, the computed curve estimates the load settlement relationship somewhat conservatively. The close agreement between the two curves indicates the reliability of the load transfer data and other design factors established in this study.

6 CONCLUSIONS

(1) A significant part of the load applied to the piles bearing in very stiff to hard clay at the Hazira site is supported by the shaft friction. Consequently, the load transfer behaviour of the piles is critical.

(2) The soil pile interaction studies revealed that the peak load transfer in clays (20 to 36 KN/m^2) and in sands (80 to 105 KN/m^2) occur at a pile movement of 5 mm and 11 mm, respectively.

(3) Average load transfer factors have been established for pile design which can be taken as 0.5 for medium to stiff clays and 0.17 for very stiff to hard overconsolidated clays. The average coefficient of lateral earth pressure,K, has been determined to be 1.9 for compact sands and 1.2 for near compact sands.

(4) The ultimate tip load in very stiff to hard clays is mobilised at a tip movement of 4 percent of the pile diameter.

(5) The computed load-settlement curve derived from the load transfer data developed in this study show good agreement with the actual load movement curves from the other pile load tests.

ACKNOWLEDGEMENTS

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