CHAPTER 6

BEHAVIOUR OF PROTOTYPE PILED RAFT

6.1 GENERAL

Although small scale model studies and numerical modelling have provided considerable amount of data on the performance of piled raft, they are subjected to certain limitations and assumptions. Therefore, German code advocates monitoring the behaviour of instrumented real size piled raft foundation. Further the development on the design philosophy of piled raft as brought out in the difference is based more on the observational methods. Here the performance of instrumented prototype piled raft was monitored during the construction and post construction period and the results are back analyzed to improve upon the design philosophy and efficiency of piled raft. An ideal design of piled raft would be the one wherein the number of piles would be minimum with the raft sharing more loads under reduced settlement. Even though in few cases as reported by Katzenbach (2000) this has been achieved, they are mostly with raft thickness of 1.5m to 3m founded on deep deposits of over consolidated clay.

If the piled raft has to be used as foundation system with more freedom and confidence, its suitability has to be established for medium sized structure also with relatively smaller diameter piles and thinner raft. Its effectiveness in overall settlement reduction must be adequately studied and validated. An settlement oriented design will be more effective it is based on observational data, duly validated by a suitable analytical test. For example the results of observations made on the piled raft of Eurotheum tower was used to design the piled raft of Max Tower (Katzenbach et al 2002). This
implies that adequate data on the performance of real size foundation must be available along with analytical validation so that the data can be used as a base for the design. It is felt very much essential that such data bank must be available for every regional deposit. The study on prototype piled raft has been carried out with the above as one of the main objective of this research.

The structure monitored in this study is a twelve-storied commercial cum residential complex which is supported on piled raft foundation. The raft is reasonably thin with the piles of smaller diameter. The piles pass through soil layers that are predominantly sandy and rest on medium dense to dense sand. The observations made in the field study are analysed to understand settlement behaviour and load sharing between the raft and piles. The results thus obtained are compared with the results of numerical analysis on prototype piled raft.

6.2 PALACE REGENCY BUILDING, CHENNAI

The building adopted for this study is located at Purasawakkam in Chennai, India. The structure is a twelve storied reinforced concrete construction with a basement floor. Plate 6.1 shows the elevation of the structure, as completed. The weight of this concrete building on the raft is 99000kN. The ground area of the building measures 32m x 25m. The height of the building is 36m. The slenderness ratio of the building if the lower dimension alone is considered works out to 3; but if overall dimension is considered it is 1.5.

The basement, ground floor and the first floor are for commercial use and the other upper floors are for residential requirement. The maximum and minimum loads on columns are 2870kN and 1050kN respectively at the raft level. The structural design has been done in accordance with the provisions of the latest Indian Standard code of practice. The minimum grade of concrete is M20 and all the reinforcements were Fe 415 grade bars. The entire building was completed in 460 days and was fully occupied shortly
thereafter. The structure was analysed assuming that it is resting on an unyielding support using STADD PRO, a versatile package for structural analysis and design. The support reactions from the analysis have been taken for the design of foundation.

6.2.1 Soil Profiles of The Building Site

Palace Regency building site is located in the busy commercial area of Purasawakkam covering over an area of roughly 4000m². This area is populated with multi-storeyed structures and most of them were supported on pile foundations. Therefore, it is decided to explore this site through three deep boreholes in order to obtain the knowledge on soil condition. Soil investigation work was carried out as per BIS specifications under the supervision of the author. In order to obtain relative strength of the strata, SPT was conducted at regular intervals.

Plate 6.1 Elevation of Palace Regency Building, Chennai
Figure 6.1 presents the typical soil profile of the site. The soil profile at the site comprised of medium stiff sandy clay layer of CI type for the top 7m depth. This layer recorded low N 2 to 4 value particularly at depth between 4m and 7m. From 7m to 14m the layer is clayey sand SC/SM type with sand content more than 65%, and the state of compaction is medium dense. Beyond 14m the percentage of sand content increases with depth and so the state of compaction. This layer extends up to 24m. The stratum beyond 24m is disintegrated rock. The bore hole was terminated in this layer, after ascertaining that the disintegrated rock extends for a larger depth. Figure 6.2 presents the sectional elevation of the structure with the soil profile and the essential soil parameters.
Figure 6.2 Sectional Elevation with Geotechnical Data
6.2.2 Selection of Design Parameters and Design of Foundation System

Since the structure has a basement it was proposed initially to support it on raft. Computations showed that the raft will undergo a total settlement of 300mm including long term settlement which is more than the permissible value as per the code IS1904-1986. Further the structure has a central courtyard and at the column loads are less in this area. This has resulted in higher differential settlement. Hence keeping the variations of column loads and the magnitude of differential settlement in mind, it was decided to support the structure on deep piles. Further few structures in the neighbouring area within 100m carrying load lesser than this structure are supported on piles. However at this stage the option of piled raft was thought of keeping the basement and the thickness of the basement raft required in mind

To the best of our knowledge no clear cut procedure was available relating to the load sharing behaviour of piled raft, of the type with thinner raft and smaller dia piles. Moreover the piles pass through soil layers that are predominantly sand. However, the principles outlined by Burland et al (1977), Padfield and Sharock (1983), and Franke (1993), were taken as the basic guidelines for the design. According to Burland (1977) the piles have to be ductile. Franke (1993) had indicated that the base resistance of the pile has to be much smaller than the shaft friction and the load shared by the raft has to be considerable. Padfield and Sharock (1983) suggested that shaft friction has to be fully mobilized for settlement reducing piles. Accordingly the main principles considered in the design are

(i) the applied load would be shared by the raft and the piles equally.

(ii) the piles have to be dominantly floating and

(iii) the settlement level must be such that the pile must mobilize the friction entirely.
While the first two aspects could be taken care of, the movement required for the mobilization of the entire friction could not be assumed. Hence it was considered that the piled raft settlement shall be more than 12mm for the complete mobilization of frictional resistance, which is the limiting settlement for individual piles as per IS 2911 (Part IV) 1985.

As large diameter bored piles were becoming uneconomical for the intensities of column load, smaller diameter piles were used. The layout of piles was made in such a way that it follows a known pattern. Among the various patterns, it was decided to locate the piles below the column as adopted by Yamshita et al (1994). It was also decided to adopt the load sharing data available under identical circumstances. To the best of our knowledge no published data was available to establish the load settlement or load sharing behaviour of piled raft, particularly for relatively medium sized building with flexible raft.

Yamashita et al (1994) established the load sharing behaviour by adopting thinner raft with smaller dia piles and showed that the load sharing between raft and the pile was in the ratio of 49% and 51% respectively for the raft with piles below the column. Based on this, the load shared by the piles and the raft was assumed as 50% of the total load in the present design. The piles were designed to have a factor of safety of 1.75, keeping in mind the likely variations of the loading in the commercial floors. The raft was designed as a flat slab treating the piles as column; 600mm was the raft thickness required from the bending moment and shear considerations for the above assumption. The layout of piles is given in the Figure 6.3.

The most important parameter in the design of piled raft is the $E_s$ value of the supporting strata. When the stratum becomes predominantly cohesionless the most reliable method to assess the soil parameters at its in-situ conditions is the N value. While a lot of empirical expressions are available to relate N and $E_s$, the charts published by Mori (1965) for clay and sand were used in this study to obtain the $E_s$ values of various layers of the deposits of this area (Appendix 2).
Figure 6.3 Lay out of piles and settlement markers
The other important factor is the load sharing ratio between the piles and the raft which was taken as 50% as stated earlier. In designing the pile group the factor of safety against block failure was computed as given by Poulos (2001)

\[
F = \frac{P_w + NP_i \eta}{P}
\]  

(6.1)

where

\begin{align*}
F &= \text{Factor of safety against block failure} \\
P_w &= \text{bearing capacity of raft} \\
N &= \text{number of piles} \\
p_i &= \text{individual pile capacity} \\
\eta &= \text{group efficiency} \\
P &= \text{total structural load}
\end{align*}

Accordingly the number of piles required were computed as 86. However the column layout and optimization of column load for pile grouping necessitated the provision of 93 piles as shown in the layout Figure 6.3. The piles provided were of 600 mm diameter under each column. Under many columns two piles were installed. The piles were installed adopting the rotary drilling equipment and were terminated in sand layer at a depth of 17m below the existing ground level. It was ensured that the bottom of the bore was clean so that the seating of the pile would be done on the natural soil. The piles were terminated in sand layer where the observed N-value was around 40. The construction of pile foundation and raft elements are presented in Plate 6.2.
6.2.3 Instrumentation of the Piled Raft and Measurements

Since the primary aim was to study the load settlement behaviour of the piled raft, importance was given to obtain the settlement values at various locations at every stages of loading. Hence settlement markers were placed at 15 points as shown in Figure 6.3. The settlement markers comprised of 75mm × 75mm × 6mm plate two numbers separated by a distance of 600mm and was made to form an open box by welding the plates with 4 bars of 12mm diameter. This box was welded to the bottom layer of the raft reinforcement. The verticality of the marker and the level of the top surface were checked using mercury levels and plumb bob. The selection of the location for the settlement markers was done in such a way that the settlement profile of the raft can be plotted in both the directions at various sections. In order to measure the settlement a standard bench mark was established such
that it can be viewed from any point and will not undergo any movement. Plate 6.3 presents a typical settlement marker in position.

Plate 6.3   Typical settlement marker in position

Plates 6.4 and 6.5 illustrate the field settlement measurements using precision level. The initial reading was taken as soon as the raft was cast. Subsequent readings were taken each time the slab was cast (i.e. immediately after the deshuttering of the slab was done). The settlement recorded by all the markers was monitored for the entire construction period of 360 days. The top most slab namely the Lift Machine Room was cast after 402 days pointing the completion of the structure, and by the time 50% of the brick work and partition along with the flooring was completed. Settlement of the raft was monitored continuously and reading were taken at an interval of every three months spread over a period of one year. During this period the rate of increase of the settlement was very small and gradual. Thereafter the balance work relating to other interior and occupational loadings were completed.

The observations were continued and completed on 796\textsuperscript{th} day from the date of commencement of construction. The period from 360\textsuperscript{th} day and 796\textsuperscript{th} day has been termed here as post construction period. The increase in the settlement was recorded at various locations regularly as far as possible, even though the construction procedure and interior work caused a lot of disturbances in the observations of few gauges. The settlement readings were
recorded with a high precision levelling instrument. The settlements recorded at various locations during the entire construction and post construction period are presented and discussed in the investigation.

Plate 6.4 Settlement measurement at one of the settlement markers

Plate 6.5 Settlement measurement using precision level
6.2.4 Load-Settlement and Load Sharing Behaviour

The readings recorded at the locations of settlement gauges are reduced corresponding to the level of permanent benchmark and the difference in level between the initial and subsequent readings are reported as settlement. Fig. 6.4 indicates the sequence of loading with time and observed settlement pattern in all the gauges representing the four tower and the central courtyard areas. The settlement readings are also presented in Tables 6.1 a and 6.1 b for three different sections. Section 1 is along the grid p, which is designed as top row. The gauges installed in this row are designed as a, f, g, m and n. Section 2 is middle row, which is close to grid ‘G’. Gauges installed along this section are 6 in numbers. They are e, b, j, g, l and o. Along the section 3 (bottom row) number of gauges are 4 (c, d, k and p).

As stated earlier the construction was completed in 360 days, and whatever settlements recorded within this period was due to self-weight of the structure inclusive of construction load. The settlement recorded after this period was post construction settlement which includes the effect of all the loads existed at that point of time. However the load variation during the post construction period is considered small.

Settlement of the raft at all the three sections followed almost identical trend with loading sequence. Among the sections, the settlement along the middle row is shifting either than other two rows. Further variations in settlement between the gauges of top and bottom rows are lesser than the gauges of middle row.
Figure 6.4  Time dependent load settlement curves
### Table 6.10 Observed settlements - Longitudinal section

<table>
<thead>
<tr>
<th>Days</th>
<th>Grid P (Upper)</th>
<th>Grid C (Middle)</th>
<th>Grid E (Lower)</th>
<th>Stages of Construction</th>
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</thead>
<tbody>
<tr>
<td>91</td>
<td>3 3 2 0 3</td>
<td>3 3 1 0 1 3</td>
<td>1 3 0 2</td>
<td>III Floor</td>
</tr>
<tr>
<td>143</td>
<td>3 3 2 3 4</td>
<td>3 3 2 2 3 5</td>
<td>2 3 2 3</td>
<td>VI Floor</td>
</tr>
<tr>
<td>204</td>
<td>4 3 3 4 4</td>
<td>4 4 2 3 4 5</td>
<td>3 4 3 5</td>
<td>VII Floor</td>
</tr>
<tr>
<td>236</td>
<td>5 4 4 4 4</td>
<td>5 6 3 3 4 6</td>
<td>4 6 3 5</td>
<td>VIII Floor</td>
</tr>
<tr>
<td>312</td>
<td>7 7 6 6 6</td>
<td>9 9 4 5 6 7</td>
<td>7 8 5 7</td>
<td>X Floor</td>
</tr>
<tr>
<td>360</td>
<td>9 9 8 8 8</td>
<td>10 11 5 6 8 9</td>
<td>9 10 7 9</td>
<td>Completion</td>
</tr>
<tr>
<td>402</td>
<td>9 9 9 10 9</td>
<td>10 11 8 6 9 10</td>
<td>9 11 8 10</td>
<td>Post Construction</td>
</tr>
<tr>
<td>796</td>
<td>12 12 12 11 11</td>
<td>13 14 11 9 11 12</td>
<td>12 13 10 12</td>
<td>Post Construction</td>
</tr>
</tbody>
</table>

### Table 6.1b Observed settlements - Transverse section

<table>
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<tr>
<th>Days</th>
<th>Grid16</th>
<th>Grid15</th>
<th>Grid7</th>
<th>Grid2</th>
<th>Grid1</th>
<th>Stages of Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>91</td>
<td>3 3 1 3 3 3</td>
<td>2 0 0 0 1</td>
<td>3 3 2</td>
<td>III Floor</td>
<td></td>
<td></td>
</tr>
<tr>
<td>143</td>
<td>3 3 2 3 3 3</td>
<td>2 2 2 3 3</td>
<td>4 5 3</td>
<td>VI Floor</td>
<td></td>
<td></td>
</tr>
<tr>
<td>204</td>
<td>4 4 3 3 4 4</td>
<td>3 3 3 4 4</td>
<td>4 5 5</td>
<td>VII Floor</td>
<td></td>
<td></td>
</tr>
<tr>
<td>236</td>
<td>5 6 4 4 6 6</td>
<td>4 3 3 4 4</td>
<td>4 6 5</td>
<td>VIII Floor</td>
<td></td>
<td></td>
</tr>
<tr>
<td>312</td>
<td>7 9 7 7 9 8</td>
<td>6 5 5 5 6 6</td>
<td>6 7 9</td>
<td>X Floor</td>
<td></td>
<td></td>
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<tr>
<td>360</td>
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<td>8 6 7 8 8</td>
<td>8 9 9</td>
<td>Completion</td>
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<tr>
<td>796</td>
<td>12 13 12 12 14</td>
<td>13 12 9 10 11 11</td>
<td>11 12 12</td>
<td>Post Construction (Final)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
During the initial sixty days practically there was no settlement or the magnitude was not measurable. This may be due to the fact that the intensity of loading was inadequate to cause any measurable movement. But when the third floor slab was cast and deshuttered, the settlement at various locations as shown by the gauges varied from 1mm to 3mm. Measurable variation in settlement occurred after the completion of the sixth floor. The corresponding load on the foundation system was around 45% of the total load. Thereafter the settlement rate was more with load and at the time of completion of structure the maximum settlement in the tower area was 11mm.

Typically the gauges b, e, c & d representing one of the tower areas showed a settlement of 11mm, 10mm, 9mm and 10mm respectively, indicating almost uniform settlement. In the central courtyard represented by the gauges “h, j and g” the recorded settlements were 6mm, 8mm, and 8mm respectively. However a gradual increase in settlement was observed and recorded after the construction period of 360 days at all the locations. The maximum and minimum settlement recorded is 11mm and 6mm at the end of construction, which are at locations b and h (central row) respectively. The gauge b represents the centre of the tower area whereas the gauge h represents the court yard area which is the least loaded area of the structure. Moreover piles provided in this area are identical to the piles provided at other areas of the raft, which apparently has reduced the settlement. Study on settlement profile showed that the settlement at centre part of the raft in each of the tower area is higher than at the edges.

The higher settlement at the middle portion is attributed to heavier column loads and the flexible nature of the raft. Measurement of settlement was continued after the completion of building. The maximum increase in settlement during the post construction period of 436 days was 3mm. The rate of increase in the settlement during post construction period is very small and gradual.
Further it can be seen from the Figure 6.4 that the rate of settlement increased during the construction period between 200 to 360 days, where the percentage of loading increased steadily from 45% to 95%. In other words we can infer that only after 50% of the structural load acted on the piled raft, the total friction got mobilized causing a larger movement of the system. This increase in the settlement indicates that the load shed by the piles is taken by the raft. It is at this stage the piles take up the role as settlement reducer.

Figure 6.5 presents the load taken by the raft at various stages of construction. Even though no instrumentation could be done for measuring the load, the load taken by the raft could be computed from the settlement observed during various stages of construction. The load shared by the raft was back calculated from the settlements measured using elastic equations (Hemsley, 2000). The tributary area of the particular column of influence of the load was computed and the stresses were calculated using observed settlement adopting elastic theory. This construction was carried out for various column sections. Since the structure is symmetry and the settlement is uniform, calculation on various sections showed almost identical trend in load sharing. For an average settlement of 3mm the load taken by the raft is of the order of 7.6%. This indicates that in the initial stages the piles take the applied load.

![Figure 6.5 Percentage Load Taken by the Raft at Various Stages of Construction period](image-url)
The steep increase occurred in the load taken by the raft occurred when the applied load was 60% of the total load. At 50% and 80% of the total load the raft was found to share 17.5% and 33% of the total load. At the final loading the raft takes 43% of the applied load.

During the initial stages of loading, most part of the load has been taken by the piles, but as loading increased the raft started sharing the load. This is clearly seen from the Figure 6.5. Sharing of load by the raft increases with increase in settlement of the piled raft system. When the load is 80%, the raft shared around 33% of the total load. However at the final load, the load taken by the raft was 43%, which is close to the assumed load share of 50% considered in the design of piled raft. Yamashita et al(1994) have also reported that the raft carry 49% of the total load which is in close agreement with the observations made in this study.

6.3 NUMERICAL ANALYSIS

The piled raft foundation of Palace Regency building was modeled numerically and the results obtained were compared with the field test results. The numerical analysis was performed by using program code ANSYS and the details of which have already been introduced. The analysis shown in the present work are restricted to elastic condition for the following reasons:

(i) From practical point of view, elastic analysis is simple and most convenient for the piled raft system which involves complex interaction between the raft, pile and soil, in addition to the three dimensional nature of foundation.

(ii) The strain level around piled foundations is small, particularly under working loads. Thus majority of piled raft foundations can be delt with using elastic approaches with moduli chosen with due consideration of small strain condition.
(iii) Elastic methods (ex: Poulos and Davis, 1980) contributed significantly to the practical analysis of different foundations. Further most of the references available on the behaviour of piled are based on elastic approaches.

Three dimensional finite element analysis was performed using ANSYS program. In order to generate the model, the principles of solid modelling have been used. The soil has been modelled with eight nodded brick elements with each node having three degrees of freedom. The raft and the piles were modelled with Solid45, which is an eight nodded brick element having three degrees of freedom at each node. The features of the solid45 elements are discussed elsewhere in the previous chapter. They are nodal translations in the x, y and z directions. The interface characteristics between the raft and soil have been represented by the element Targe 170 and Conta 174.

In the analysis perfect contact between the raft and soil is ensured through default option available in the program. Half model has been taken to reduce the computing time. Certain amount of marginal variation found during the idealization has been ignored while generating the model, as this will not affect the performance of the model during the analysis and accuracy of the results. Appropriate boundary condition has been imposed on the edges of the model. In order to generate the mesh, map meshing technique has been adopted. Reul and Randolph (2002) have studied the effect of mesh refinement on the quality and accuracy of the results and have proved that mesh refinement beyond a certain extent does not enhance the quality of the results. However in our analysis the maximum aspect ratio adopted was 5. The simulated finite element model of piled raft is shown in Figure 6.6.

The extent of soil medium and element sizes were chosen by trial and error to suit the required accuracy and computing time. It was found that for the breadth of soil medium more than 2 times that of the least width of foundation the variation in settlement and contact pressures are maximum.
Therefore soil medium considered in the half model extends 30m in the x direction and 30m in y direction. The model has 93,000 elements and 108761 nodes.

Figure 6.6 Finite Element Simulation and Meshing of Piled Raft

6.3.1 Material Property

There are only two materials namely concrete and the soil. The property of the concrete namely the elastic modulus was taken as per the recommendation of IS 456-2000. In the case of soil the elastic modulus of the soil $E_s$ was found from the N values. The properties of materials used in the ANSYS analysis are as presented in Figure 6.7. The Poisson’s ratio of 0.35 was used in the analysis for all the soil layers and for the concrete the value adopted is 0.20.
6.3.2 Application of Load

Since the structure is a tall frame, a three dimensional frame analysis was performed for various load combinations and support reactions at column points were arrived at. For the foundation design these support reactions were taken. The column forces (axial, lateral and moment) computed from the three dimensional frame analysis in the form of support reactions were applied at the respective column locations. Since the contribution of transient loading namely wind and earthquake in the form of horizontal loads are beyond the scope of present study, these forces were not accounted for in this analysis. Even though the raft was placed at a depth of 3m below the original ground level, the change in the in-situ stresses due to excavation was not simulated in the present study.

6.3.3 Settlement of Piled Raft

The output from the analysis pertaining to the settlement behaviour was studied from the settlement contour and the results are shown in Figure 6.8.
The nodes representing each settlement marker was located and the magnitude of settlement was picked up. These values were compared with the final settlement measured along the longitudinal (grids P,B,&G) and transverse sections (B14-P14) 10 Figure 6.9. Since the structure has symmetry in transverse direction one section was considered adequate to represent the transverse behaviour. The measured settlement along any section shows that the settlement at edges is more than the centre and the maximum difference is around 4mm. However it is to be noted the central area is courtyard which is only 3 storied. The column load is relatively smaller which justifies the reduced settlement. This indicates that the differential settlement is far less than the permissible value. Further the settlement at the edge and centre of the piled raft are 6% and 7% respectively of the plain raft elastic settlement (computed). Thus the piled raft reduced the settlement effectively despite adopting a raft of 600mm thick (flexible raft).
Figure 6.9  Observed Settlement Vs Computed Value at Various Section

A comparison of the observed and computed settlement profile indicates that the results agree well. However, the computed values are marginally higher in general. The computed settlement is higher by 2.5mm than the observed values in the edges and the same is lesser by 3.5mm in the mid portion of the raft. This is perhaps in reality the structure had a retaining wall which was not modeled in the numerical model. The presence of retaining wall increased the stiffness of the raft causing a reduction in the settlement. In the central portion the computed settlement was less than the observed value. This is perhaps the contribution of super structure rigidity was not taken in the numerical modelling. Also in elastic analysis irrespective of the relative stiffness of soil-foundation system and types of soil, the contact pressure is always higher (manifolds) at the edges than at central part of the
raft. But this difference reduces in the central grid B. In grid G the computed values are lower than the observed value.

The results in the central portion where the courtyard has been located agree very closely. Here the loadings are relatively smaller, indicating that at lower level of load the analytical and the observed values agree very well. The computed and the observed results agree to a reasonable extent in the transverse sections also. Looking at the degree of agreement it can be stated that the numerical and prototype result compare closely for linear elastic condition of soil. Since most of the settlement has taken place during the construction and after post construction increase in settlement is very less, we can say that under working load the load settlement relation is close to elastic behaviour of soil.

6.3.4 Load Sharing Between the Raft and Piles

In Figures 6.10, 6.11 and 6.12 the contact stress at selected points are presented. The stress thus obtained along specific sections are presented and discussed below. Figure 6.13 presents the contact pressure along the grids P, G and B. These are typical grids having the piles. The stress has been picked up close to the pile location and in between the pile location along the grids. Each grid presents contact pressure over half section of the raft as the either half is symmetrical with each other. The peak values indicate the stress near the piles and the lower values indicate the stress in between the piles. In the case of grid P the raft stress is maximum at the edge; the magnitude being 0.12N/mm$^2$. The peak value varies from 0.10N/mm$^2$ to 0.12N/mm$^2$. In the grid G the edge stress is 0.07N/mm$^2$ and the peak values various from 0.09N/mm$^2$ to 0.10N/mm$^2$. In grid B the edge and the peak value are more or less uniform with the magnitude being 0.1N/mm$^2$. The low values which are in between the pile group indicate an average value of 0.05N/mm$^2$ to 0.06N/mm$^2$. The grid P as can be seen from the pile lay out has a tributary
area of raft much lesser compared to the other two grids. This is the outer most grid having a higher load due to the presence of the wall for the entire height and RCC retaining wall from the ground to the basement level. Hence with a higher loading and lesser raft tributary area, higher percentage of load gets transferred to the piles and the load taken by the raft becomes less.

In the case of grid G and grid B, the tributary area of the raft is higher and the raft shares a higher amount of load. A study of the curves clearly indicates that the magnitude of the lower peak remains more or less equal.

The peak value near the pile location ranges from 0.1N/mm² and the lower peak value ranges between 0.04 to 0.6kN/mm². This indicates that irrespective of the location the magnitude of the contact stress is nearly equal indicating that the contact pressure is uniform.
Figure 6.11 Raft contact stress along grid G

Figure 6.12 Raft contact stress along grid B
Figure 6.13  Contact stress at specific points of the raft
Figure 6.14 Contact stress between the rows of piles in transverse sections

Figures 6.14 presents the raft – soil contact stress variation in other raft sections, which are between the pile grids and along the transverse section. The first section marked, is section between grids P and L taken approximately at a distance of 1.5m down from the grid P. The raft has an edge stress of 0.06N/mm². The average contact stress on the raft found out to 0.06N/mm², with a variation being from 0.04N/mm² to 0.07N/mm². The
second section marked between grid G and H lies exactly in the middle of grid G and H. In this case the edge stress is in the order of 0.055N/mm² and the stress along this section is uniform than other sections. The third section is between B and C grids which is located at a distance of around 1.4m from the grid B appears towards C. At this section, the edge stress is higher which is of the order of 0.061N/mm² and the average stress is 0.055N/mm². The transverse section taken in between the grids 13 and 15 indicates a similar trend with relatively uniform stress of 0.045N mm². As can be seen in all these grids, the raft contact stress is fairly uniform and of the order of 0.055N/mm² which is 38% of the applied load.

![Figure 6.15 Typical Pile numbering](image)

The load shared by the pile group has been arrived from the average vertical stress values of pile elements. Figure 6.15 presents the pile layout considered for picking up the stress on the pile to compute the head
load and tip load of the pile. Very small areas of stress concentrations have been ignored, stresses at the head and tip of certain piles are presented in Figure 6.16 and 6.17 respectively. From the figures it can be seen that the stress in the pile is decreasing towards the positive gradually. The decrease in stress with depth on the pile is due to frictional resistance offered by the soil around the pile. The average of remaining values of stress multiplied by the area has been taken as head load and tip load.

**Figure 6.16** Typical head stress values

**Figure 6.17** Typical tip stress values
The head load and tip load distribution with the column load has been plotted in the form bar chart as shown in Figure 6.18. In general the percentage of tip load varies from 20% to 40% but in majority of the cases the variation is 20% to 30%. Further it is noticed that the tip resistance of piles in the central part of tower sections is more. It is because of more confinement to these piles than at the edges. This is also quite evident from the section B, where the tip resistance of the piles are the lowest. The magnitude of the tip load clearly indicates the ductile behaviour of the piles as expected. Correspondingly the raft has taken a reasonable share of the load.

Figure 6.18 The Head Load – Tip Load Distribution with the column load (continue…)
Hence it can be conclusively said that the behaviour of the foundation system has been as expected with the piles behaving as a flexible group and settlement reducer. Though the piles have been located below the columns, the load sharing of the raft has been 43% from the observed settlement value and 37% as per the numerical modeling with the piles.
behaving as a settlement reducer sharing 57% to 63% of the total applied load.

6.4 SUMMARY

The study done on the prototype piled raft has indicated that the foundation system can be used successfully for moderately loaded structures also. It has shown that the major portion of the settlement takes place during the construction period itself. The settlements measured at field and the values obtained from the numerical analysis agreed closely indicating this, even linear analysis can predict the behaviour of the piled raft so long as the settlement is elastic settlement. The load sharing between the pile and the raft had been 57% and 43% as observed. This indicates that the design compares well with the third generation piled raft as qualified by Katzenbach. The numerical model predicted relatively higher settlement and lower value for the load shared by the pile group.