INVESTIGATION INTO THE DISTRESS FAILURE OF STRUCTURES AT MANALI, CHENNAI

A. Amala Raju Arul
Post Graduate Student, Department of Civil Engineering, Anna University Chennai, Chennai-600 025, India.
E-mail: amalarajuarul@gmail.com.

K. Premalatha
Assistant Professor, Department of Civil Engineering, Anna University Chennai, Chennai-600 025, India.
E-mail: kvprema@annauniv.edu

ABSTRACT: The present study is carried out in order to evaluate the possible causes of distress in the ‘Sites and Service Scheme’ complex, Manali, Chennai, by conducting laboratory and field investigation. Laboratory test are carried out on disturbed and undisturbed soil samples obtained at the site of damaged buildings. The causes of failure of the structures are picturised taking into account the soil properties, intensity of loading, nature of foundation and pattern of cracks developed.

1. INTRODUCTION

Over the years “distress failure” of structure, and in particular, lightly loaded structures have been reported from several parts of the city of Chennai and now it is established that a major part of the city of Chennai is underlined by the potentially expansive silty clay. The X-ray diffraction tests have also identified the presence of the active clay minerals such as illite and montmorillonite in these clays.

Problems associated with the design of foundations for residential and other light structure in the city of Chennai have been dramatically highlighted in recent years by wide spread damage particularly by following periods of severe drought. The moisture condition of Chennai city is widely varying. The average annual precipitation is around 1220 mm. During 1968, 1974, 1983, 1992 etc., there was deficiency in rainfall, and the distress cycle in buildings was repeated.

2. DETAILS OF THE DISTRESSED COMMUNITY FACILITY BUILDINGS

This paper presents the investigations carried out to identify the causes of the distress and to recommend suitable remedial procedure for site and service scheme complex of Manali. This scheme comes under the urban development project-II of Chennai city. This particular area lies in between Manali new town and MRL Campus.

The site and service scheme has covered a total area of 79.17 hectares and divided into two phases (Phase I and Phase II). A total number of 5656 residential plots were proposed in the above phases as 2929 and 2727 respectively. The residential plots were planned for all sort of people of society like, High, Medium, Low income group. Apart from the residential plots a number of community facility buildings like fire station, clinic, primary schools and high schools were also planned.

Nearly 27 numbers of buildings of different category are completed during the year 1988. The scheme was executed partly in a low lying area, which was filled up to the level of higher elevation. The details about the filled up earth was not known. Also it was mentioned that during 1991, due to heavy rainfall the entire area of the scheme was completely covered with water and the elevation of the water table above the ground level was about 1m, and that of floor level was about 0.54 m.

Finally the collected data indicated that the first crack in the completed community facility buildings was developed during the year 1992.

3. FOUNDATION DETAILS

Most distressed structures are single storied lightly loaded structures. To examine the causes of distress it is necessary to know about the foundation details of the structures and the loadings intensity on the structures. Hence the foundation details and the loading intensity of the different structures are collected.

The loading intensity of the different structures is primary school (Phase I & II) – 50 kN/m², fire station (Phase I) – clinic (Phase II) 30 kN/m² and the high school, maximum load on the column is 240kN.

The depth of the foundation of the lightly loaded structures is 1.4 m to 1.6 m. The type of foundation adopted was conventional reinforced strip raft for all lightly loaded structures and isolated column footing for high school building. It was observed from the foundation details, the foundation bed of fire station and police station was prepared with lime sand mixture of 1:1 ratio.

The diameter of the main reinforcement bar in the beam was 12 mm and that of slab was 8 mm. the diameters of the distributors were 8 mm and 6 mm respectively. The M15 grade of concrete was used for both the slab and beam.
4. NATURE OF THE DISTRESS

As the second step of the investigation, the nature of the distress i.e. the typical crack pattern of the different community facility buildings were observed. These crack patterns were observed, during the month of September 1994. In almost all structures the crack patterns were similar; some of the typical crack pattern is illustrated in Figure 1.

This typical distress indicated the moisture variation of the covered interior area and the exposed periphery area is the primary cause of distress, and was the most sever condition.

The category of damage or degree of damage of each and every structure based on the observed crack width and number of cracks were quantified.

The scale adopted is the one suggested by Burland et. al (1978).

5. SUBSOIL EXPLORATION

To examine the causes of the distress, a detailed subsoil exploration scheme was planned and both field and laboratory investigation have been carried out.

Since the area covered under this scheme is very large, as an initial stage two static cone penetration tests were conducted. The location of the static cone penetration test selected in such a way that one at the zone of very slightly distressed structure (location $P_1$) and the other at very slightly distressed structure (location $P_2$).

To investigate the properties such as, the grain size distribution, consistency limits, shear strength parameters and swell pressure, differential free swell for disturbed and undisturbed representatives samples were selected.

The bore log characteristic of clinic, phase–II is presented as Table 1.

6. INTERPRETATION OF FIELD AND LABORATORY TEST DATE

The static cone penetration at location $P_1$ and location $P_2$ represents the variation of the strength of the soil between the two locations.

### Table 1: Bore Log Characteristic of Clinic, Phase – II

<table>
<thead>
<tr>
<th>Soil Profile of Location 3 (Clinic Phase-II)</th>
<th>Grain Size (%)</th>
<th>Consistency Limits (%)</th>
<th>Plasticity index</th>
<th>I.S. classification</th>
<th>In-situ wet density (g/cm³)</th>
<th>Natural water content (%)</th>
<th>Unconfined Comp. strength (Kg/cm²)</th>
<th>Undrained cohesion (Kg/cm²)</th>
<th>Angle of shearing resistance (deg)</th>
<th>Differential free swell index (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–1.5 m, Light Brown Soft Clay</td>
<td>– 9 50 41 57 25.6 11.5 31.4</td>
<td>CH 1.78 – 0.39 0.195 – 50</td>
<td>0.195 0.185 – 50</td>
<td>50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5–3.6 m, Brown Soft Clay</td>
<td>– 3 39 58 56 25.6 12 30.4</td>
<td>CH 1.54 54 0.37 0.185 – 50</td>
<td>50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.6–5.4 m, Dark Gray soft clay with fine sand</td>
<td>– 56 13 31 34 20 18 14</td>
<td>SC – – – – – – –</td>
<td>–</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The average cone resistance at different depths of location $P_1$ and $P_2$ are given in Table 2.

<table>
<thead>
<tr>
<th>Location $P_1$</th>
<th>Location $P_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>qc, cone resistance, (Kg/cm²)</td>
</tr>
<tr>
<td>0–1.7</td>
<td>18</td>
</tr>
<tr>
<td>1.7–3.7</td>
<td>2</td>
</tr>
<tr>
<td>3.7–7.4</td>
<td>18</td>
</tr>
<tr>
<td>7.4–9.5</td>
<td>15</td>
</tr>
<tr>
<td>9.5–11.5</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>6.3–9.0</td>
</tr>
<tr>
<td></td>
<td>9.0–11.5</td>
</tr>
</tbody>
</table>

The static cone resistance profile obtained at location $P_1$. Primary school-6 phase–II is the worst profile. The degree of the primary school-6, phase–II is very severe, where as the degree of the distress of primary School Phase I is very slight. Taking the worst cone resistance profile in to consideration, the allowable bearing capacity of the soil for different dimensions of strip raft (fire station, clinic), primary school and for the framed high school buildings column of maximum dimensions are worked out. The net allowable bearing capacities are estimated using IS-6403-1978, code of practice for shallow foundation. Similarly, the settlements of the foundations are arrived based on IS. (8009 part II 1976) code of practice for shallow foundation settlement evaluation. The net allowable bearing capacity of primary school (phase–II) is around 12.5 kN/m². To evaluate net safe bearing capacity, a factor of safety of 2.5 is used. The actual load intensity of primary school is 50 kN/m² which is nearly 4 times the net allowable bearing capacity. The evaluated settlement for the load of 50 kN/m² is around 34 mm, which is found to be greater than 25mm, the allowable settlement. Similarly, the net allowable bearing capacities of fire station and police station, worked out 13.2–13.6 kN/m². The net safe bearing capacity of high raised framed structure high school building is 25 kN/m² nearly twice of the allowable. The settlement of this high raised structure worked out to be 24 mm, which is within the permissible limit.

There is a possibility for differential settlement of the structures in both lightly loaded, load bearing wall structures and high-raised framed structures. But the rigidity of the high-raised framed structures is more than that of load bearing wall structures; the possibility for distress for framed structure is relatively less than that of the load bearing wall structures. Also this prevailing data indicates that there may be a possibility for the low bearing capacity. The primary school (phase–II) foundation directly rest on the soft clay layer where the cone resistance is only 20 kN/m².

The static cone resistance within the depth of influence of the foundation is also found to be low. Hence from the field test data it is observed, the failure of the structure or the distress of the structure may due to the low bearing pressure value.

The classifications of the soils were done based on IS classification. Apart from this soil was also classified based on different classification system such as Donaldson, Holtz and Gibbs, IS 1498, IS 2720, Seed et al and Ranganthan & Satyanarayana to identify the volume change potential. The volume change potential of the soil is high to very high.

Also it is identified that in almost all bore hole, the soil up to the depth of around 2.5 to 3.5 m was high to very high expansive. Below 3.5 m depth relatively a low swelling soil layer is identified. But in the location 2, primary school–6 phase–II, it was identified the soil upto the investigated depth of around 4.8 m is medium to high expansive.

The laboratory swell pressure test was carried out to determine swell pressure and % swell. The dry density of the soil is also worked out to be 13 kN/m², which is the lowest density for on land deposits.

The unconfined compression test conducted on the collected undisturbed soil sample also indicated a very low value of undrained cohesion (6.5 kN/m²) confirmed the presence of the soft clay layer at depths of 1.6 m.

The laboratory test data also prevails, the distress of the structure may be due to the swelling and shrinking characteristics of the under lying soil.

The observed crack pattern also resembles with previous case studies which are identified as the expansive soil crack patterns. This cracks might have been occurred due to the variation of moisture in the covered area of the building and the periphery area of the building. This peripheral shrinkage was predominant in these distressed structures. The peripheral shrinkage is explained by number of phenomena, such as the evaporation and transpiration of moisture. Similarly the swelling of the interior parts of the structure was explained by the accumulation of moisture due to thermo-osmotic film flow and soil suction forces.

The development of the resultant ground curvature is responsible for the production of the distress effects, which is the resultant effect of bending and torsional twisting of the foundation. The extent of which a structure can conform to a subsidence profile is the foundation of its flexibility. High degree of flexibility didn’t permit the foundation to redistribute supporting reactions. Hence the structures get distressed due to the ground movements.

The effect of vegetation also plays a major role in the peripheral shrinkage. Almost all area of the structure is surrounded by Presopis Juliflorea tree, which will extract a large quality of water from the clay soil for their growth, the roots of the trees that are spread horizontally, which in turn exert enormous pressure during the process of growth on the member above, resulting in the development of cracks. In
Investigation into the Distress Failure of Structures at Manali, Chennai

primary school-2 and primary school-6 (phase II) rainforests (Akbeizia saman) was found near the northern and southern corner.

7. CONCLUSION

The various causes for distressed lightly loaded structures at Manali were investigated through field and laboratory testing. The causes for the distress are identified as (i) presence of soft layer within the significant depth of influence (ii) non-uniform load distributor of columns (iii) expansive nature of the soil (iv) presence of moisture loving trees such as Prosopis Juliflorea and Akbeiziasannan (Rain tree).

Finally based on the degree of distress of the different structure, certain remedial option was recommended.

REFERENCES


