SOME SUGGESTIONS FOR THE STRUCTURAL DESIGN OF BUILDINGS IN COMPRESSIBLE GROUND WITH SPECIAL REFERENCE TO HOUSING IN LOW LYING AREAS
by
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Abstract

Recommendations for the overall dimensions and planning of buildings are given considering the overall stiffness of the structure. A new design procedure has been proposed taking into account the stiffness of the structure, the stiffness of the soil, and the expected settlement of the structure. It is shown that the increase in the Relative Stiffness Factor beyond certain experimentally established values has little effect in reducing the differential settlements of housing in low lying areas. Experimental evidence is provided to show that, contrary to present assumption, masonry foundations can undergo as much total settlement as reinforced concrete foundations. Therefore, masonry foundations provide a very economical type of foundations for buildings in compressible ground. It is also shown that lateritic fills are very effective for reducing foundation stresses when compressible layers are found near the surface. However, the settlements due to the fill load could be quite high and, therefore, they must be allowed for before the commencement of construction.

1.0 Introduction

The usual method of designing buildings consists of the following steps:

1) the architect plans the building in accordance with the requirements of the client;
2) the architectural plans are handed to the structural engineer who designs the superstructure using the most suitable structural form;
3) the structural engineer designs the foundation taking into account the loads that have to be transferred to the ground, and the site investigation report which provides information on the sub-surface conditions.

Whilst this approach may be considered as reasonable for the design of buildings on 'good ground' where the settlements are small, it is shown in this paper that this method of design is inadequate for designing buildings in highly compressible ground. The objective of design from structural considerations should be to limit the deformations of the structure to a magnitude that can be safely tolerated by the structure.

2.0 Limiting Deformations

Studies of limiting deformations in buildings have been carried out in U.K. by Skempton and MacDonald (1956), Meyerhof (1956), Burland

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and Wroth (1975); in Poland by Polshin and Tokar (1957); and in Norway by Bjerrum (1963). Their studies showed that load bearing walls have a different mode of deformation from framed structures. Consequently, limiting deformation for load bearing wall structures were defined in terms of the deflection ratio ($\frac{\Delta}{L}$) as shown in Fig. 1a, and for framed structures in terms of the angular distortion ($\beta$) as shown in Fig. 1b.

**Fig. 1a** - Definition of Deflection Ratio for load bearing wall structures

**Fig. 1b** - Definition of Angular Distortion for framed structures.
e.g. Skempton and MacDonald (1956) found that the limiting angular distortion for the onset of cracking in plaster of framed buildings was 1/300; and

Meyerhof (1956) found that the limiting deflection ratio for load bearing walls was 1/2500.

A similar study was undertaken at the NERD where the settlements of a large number of buildings in the low lying areas of Colombo were monitored. Tennekoon, Sivakugan and Lakshan (1988) have reported these results which show that the limiting deformation criteria for the onset of cracking are

1) angular distortion of 1/300 for framed structures; and
2) deflection ratio of 1/2750 for load bearing wall structures.

3.0 Relative Stiffness Factor

The avoidance of differential movement in the structure involves a consideration of stifferers both of the structure and of the soil. Meyerhof (1953) defined the parameter Relative Stiffness Factor (K) as

\[ K = \frac{\text{Stiffness of the structure}}{\text{Stiffness of the soil}} \]

It is this parameter K which will control the amount of differential settlement for any given total settlement of the structure.

Tennekoon, Sivakugan and Lakshan (1988) report on a study made on the overall stiffness of structures in the low lying areas for Colombo. Using the recommendations of Meyerhof (1953), they found that the theoretically determined stiffnesses corresponded reasonably well with the actual stiffnesses of the structures as determined by their settlement behaviour.

4.0 Foundation Design using Soil Structure Interaction

Soil structure interaction is an inevitable mechanism that occurs with structures on yielding foundations. One of the objectives of soil structure interaction analyses is to estimate the form and magnitude of the relative deflections, which information is then used to assess the likelihood of damage.

Theoretical relationships, some analytical and others using numerical methods of solution, have been established between the differential settlement expressed as a ratio of the total settlement and the Relative Stiffness Factor; e.g. Meyerhof (1953); Brown (1969); Fraser and Wardle (1976); Alam Singh (1986). However, in all these solutions, the contribution of the super-structure to the overall stiffness has not been taken into account. The State of the Art Report on "Structure-Soil Interaction", ANON (1977), gives the example of an analysis carried out on a 22-storey residential block of flats founded on a 0.76 m thick raft. Satisfactory agreement between the measured and computed differential settlements was possible only after the overall stiffness of the structure was converted to an equivalent raft of thickness 4.6m. This example illustrates that detailed analyses whilst being very sophisticated and also expensive may not always give reliable answers to engineering problems.

Analytical methods of solution require mathematical modelling for the structure, foundation, and the soil. The accuracy of the results depend on how good the models are. In such a situation, experimentally obtained results sometimes gives better answers.

Using the results of the settlements measured in the buildings of the low lying areas of Colombo, Tennekoon, Senanayake and Amaratunga (1988) have developed charts relating the differential settlement (for the onset of cracking) expressed as a ratio of the total settlement to the Relative Stiffness Factor. These relationships for (1) framed structures and (ii) load bearing wall structures are given in Figs. 2a and 2b respectively.

4.1 A Methodology for design

The results of Figs. 2a and 2b may now be used to provide guidelines for the design of buildings in compressible ground. In such a design the following steps should be carried out:

1. Determine the Relative Stiffness Factor for a structure, and then from Fig. 2a or Fig. 2b, obtain the ratio \( \Delta / \rho \) for the onset of cracking.

2. Determine the expected total settlement of the structure using classical theories of Soil Mechanics.

3. From steps (1) and (2), determine \( \Delta \) for
\[ \frac{\Delta}{P} \]

**K** = Theoretically determined Relative stiffness Factor

\[ \Delta = \text{Maximum differential settlement between columns at failure, assuming failure at } \frac{\Delta}{L} = \frac{1}{300} \]

\[ P = \text{Average settlement corresponding to failure at } \frac{\Delta}{L} = \frac{1}{300} \]

**Fig. 2a:** Relationship between \( \frac{\Delta}{P} \) and K at failure, assumed to occur at \( \frac{\Delta}{L} = \frac{1}{300} \), for framed structures
$K = \text{Theoretically determined Relative stiffness Factor}$

$\Delta = \text{Maximum differential settlement at failure, assuming failure at } \frac{\Delta}{L} = \frac{1}{2750}$

$p = \text{Average settlement corresponding to failure at } \frac{\Delta}{L} = \frac{1}{2750}$

Fig. 2b – Relationship between $\left[ \frac{\Delta}{p} \right]$ and $K$ at failure, assumed to occur at $\frac{\Delta}{L} = \frac{1}{2750}$, for load bearing wall structures
the onset of cracking. Then check whether the limiting deformation criteria have been exceeded or not; i.e. for framed structures the angular distortion should be less than 1/300, and for load bearing wall structures the deflection ratio should be less than 1/2750.

4. If from step (3) it is found that cracking would occur, then increase the stiffness of the structure. This can best be done by suitably re-adjusting the plan dimensions of the structure; e.g. by breaking up a long building into several smaller ones. It is seen from Fig. 2a that for framed structures \( \Delta/\rho \approx 0.1 \) for \( K = 0.3 \); and the increase of \( K \) beyond this value reduces the differential settlement only by a small amount. Similarly, for load bearing wall structures, \( \Delta/\rho \approx 0.035 \) for \( K = 1.0 \); and the increase of \( K \) beyond this value has little effect in reducing the differential settlement.

Therefore, using the value of \( \Delta/\rho = 0.1 \) for framed structures and \( \Delta/\rho = 0.035 \) for load bearing wall structures, if it is found that the limiting deformation criteria are still exceeded, then the following alternatives are recommended for consideration:

1. reduces the expected settlement of the structure either by reducing the stresses coming on the compressible layer by using a fill material; or by improving the ground by pre-consolidation or any other economical soil improvement technique.
2. provide deep foundations.

In this method of design, it is the differential settlements rather than the total settlements which serve as the limiting deformation criterion. This is because it is the differential settlements which cause distress to a structure.

5.0 Recommendations for the Planning of Buildings from Stiffness Considerations

The preceding sections show that in computing the Relative Stiffness Factor, it is necessary to determine the stiffness of a structure as a whole rather than the stiffness of the foundation only. Although the stiffness of structural elements are often used in structural design calculations, it is seldom that the stiffness of the structure as a whole is considered in design. An exception is the design of earthquake resistant structures, and a study of the design of these structures showed that many concepts used in their design can be usefully carried over to the design of buildings in highly compressible ground.

It has been shown by Dowrick (1977) that in earthquake resistant design of buildings, some guiding principles should be followed. These are: "be simple; be symmetrical; not be too elongated in plan or elevation; and have uniform and continuous distribution of strength". It was shown that symmetry is important in both directions in plan, because the lack of it produces torsional effects which are very destructive and difficult to estimate. Again it was recommended that if a squarish plan for a building is not satisfactory, then two or more buildings with movement gaps between them should be used. Openings in brickwalls should be kept to a minimum, and they should be distributed to be as uniform as possible.

It is found that the State of the Art Report on "Structure-Soil Interaction", Anon (1977), provides similar recommendations for buildings where large settlements are expected. These recommendations are:

1) considering the plan shape of the structure, a simple compact structural unit is better able to withstand differential settlements than an elongated shape;
2) the sizes and placings of doors and windows are important;
3) re-entrant corners should be avoided.

Therefore, it is suggested, that the stiffness of buildings in marshy areas will be improved if the recommendations of Dowrick (1977) and Anon (1977) are followed as far as possible.

6.0 Design Considerations for the Foundations

6.1 Foundations for load bearing wall structures

In the structures that have been studied, three types of foundations have been used;
viz. masonry strip foundations, reinforced concrete inverted-T foundations, and the Vierendeel foundations. Presently, there is no objective criterion for the selection of the type of foundation. It is usually assumed that masonry foundations are suitable for lightly loaded structures in good ground conditions when the expected settlements are small. When the total settlements are high, the differential settlements would also increase; and it is assumed that masonry foundations are weak in bending and therefore, reinforced concrete foundations should be used. However, the experimental results of the structures studied show that masonry foundations can undergo total settlements as much as reinforced concrete foundations. For example, the settlements recorded in the different blocks of the Nawagampura Housing Scheme are shown in Table 1. The average curve showing the variation of the deflection ratio with the average settlement of these blocks is shown in Fig. 3. It was also found that distress occurred when the deflection ratio was 1/2750. It should be noted that this limiting deflection ratio is the same for both masonry foundations and reinforced concrete foundations. Therefore, the premise that masonry foundations are less suitable than reinforced concrete foundations when large settlements occur is not correct.

A possible explanation for this is that masonry foundations carry load not by bending but by arch action. Hence, it is incorrect to compare the bending strengths of masonry foundations and R.C. foundations. The limit deformation criterion for both types of foundations being the same, the method of analysis given in the previous section is applicable to both. Therefore, taking an example of two structures having the identical Relative Stiffness Factor but one having masonry foundations and the other R.C. foundations, it is postulated that both structures are equally likely to fail or not fail depending on the magnitude of the deflection ratio. In other words, rubble foundations (which are much cheaper than R.C. foundations) would work equally well as R.C. foundations, and hence they are recommended for use in housing when strip foundations can be used.

The R.C. inverted-T foundation and the Vierendeel foundation are generally stiffer than the masonry foundation. But since in the case of load bearing wall structures most of the structure stiffness is provided by the superstructure, the nett increase in stiffness due to the foundation is not very significant.

The design of the R.C. inverted-T foundation and the Vierendeel foundation has so far been done empirically. Tennekoon and Raviskanthan (1983) have shown that these foundations can be designed rationally based on limiting deflection criteria. It was shown that the longitudinal bending of these foundations can be studied on the basis of an equivalent loading system of a simply supported beam carrying a uniformly distributed load which at failure is given by

\[ W = 0.0279 \frac{EI}{L^2} \]

where \( EI \) is the stiffness of the foundation; and

\( L \) is the length of the foundation in the direction of bending.

3.2 Foundations for framed structures

In the structures that have been studied, the foundations used are:

1) Individual pad footings for the columns;
2) Individual pad footings with the columns connected by plinth beams;
3) R.C. strip footings;
4) Vierendeel foundations.

It was found that framed structures tend to be less stiff than load bearing wall structures because of the large openings that were provided in the walls for doors and windows. The contribution of the frame to the stiffness is small compared to the contribution of the infilling. Hence, the Vierendeel foundation was found to be a good method of providing stiffness to framed structures.

Again, from considerations of stiffness and the ability to resist differential settlements, individual pad footings with the columns connected by plinth beams are preferred to individual pad footings.

In structures which undergo large foundation settlements, it is very necessary to incorporate these settlements for the determination of the bending moments and shear forces in the structure. The matrix method of analysis is well suited for such an analysis, but the
<table>
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<th>Block No.</th>
<th>Range of settlement (mm)</th>
<th>$\bar{\rho}$ average (mm)</th>
<th>$\Delta$ (mm)</th>
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<td>44 - 86</td>
<td>62</td>
<td>1.5</td>
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<tr>
<td>C</td>
<td>74 - 157</td>
<td>115</td>
<td>3</td>
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<td>D</td>
<td>153 - 230</td>
<td>191</td>
<td>4</td>
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<tr>
<td>N</td>
<td>70 - 107</td>
<td>92</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Settlements recorded in the Nawagampura Housing Scheme where masonry foundations have been used.
Fig. 3  Relationship between deflection ratio & average settlement for blocks in Nawagampura Housing Scheme
foundation settlements have to be provided as an input data. Tennekoon and Raviskanthan (1988) have proposed a method for determining these settlements based on limiting deformation criteria.

7.0 The Influence of a Lateritic Fill over Compressible Layer

The commonly used method of construction of buildings in marshy lands is to place a lateritic fill at the surface prior to building. The main advantages of such a fill are:

1. it raises the plinth level so that the building is free from flooding during periods of heavy rain;
2. it provides a working platform for men and machines;
3. it reduces the stresses coming on the compressible layer to an amount within the allowable bearing capacity of the weak soil; and
4. it improves the strength of the weak soil due to consolidation of this layer.

7.1 Stress reduction in compressible layer

Studies based on elastic theory showed that a fill is very effective in reducing the stresses from the foundation. At one of the sites that were studied,

\[
\begin{align*}
\text{thickness of peat layer} & = 5.90 \text{ m} \\
\text{thickness of lateritic fill} & = 2.90 \text{ m} \\
\text{depth of footing} & = 0.90 \text{ m} \\
\text{footing dimensions} & = 1.2192 \text{ m} \times 1.2192 \text{ m}
\end{align*}
\]

Analysis showed that in this case the foundation stress of 164 kN/m² was reduced to 8.2 kN/m² at the surface of the peat layer because of the presence of the fill.

7.2 Settlements due to fill

When a fill is placed over a compressible layer, consolidation takes place in the compressible layer due to the fill load. As fill loads usually extend considerably in the lateral directions, their zone of influence is quite deep. If sufficient time is not given for the compression due to the fill to be complete prior to the commencement of building operations, then cracking may occur.

At one of the sites which was studied, 1.5m of fill was placed over an existing fill of 3m depth and building operations commenced immediately. Further, the construction was completed in a short time of 3 months. Computations showed that whereas the settlement due to the structural load was negligible, the structure would settle by as much as 225mm due to the newly placed fill. As predicted, large settlements did take place and the structures soon showed signs of distress.

Therefore, it is concluded that the use of a fill in marshy areas can be very beneficial for housing with shallow foundations provided that construction commences only after most of the consolidation settlements due to the fill are complete.

8.0 Conclusions

Design guidelines for the superstructure have been formulated based on the overall stiffness of the structure. The principles used for earthquake resistant design (from stiffness considerations) may be used to provide guidelines for the plan shape of the structure, and the planning of sizes and distribution of openings in brickwalls.

Using the measured settlements of a number of buildings in the low lying areas in and around Colombo, relationships were obtained between the ratio \( \frac{\Delta}{\rho} \) at failure and the Relative Stiffness Factor \( K \) for \((i)\) framed structures, and \((ii)\) load bearing wall structures. \((\Delta\) is the differential settlement and \(\rho\) is the total settlement). These results show that

a) for framed structures, \( \frac{\Delta}{\rho} = 0.1 \) for \( K = 0.3 \), and the increase of \( K \) beyond this value has little effect in reducing differential settlement; and
b) for load bearing wall structures, \( \frac{\Delta}{\rho} = 0.035 \) for \( K = 1.0 \), and the increase of \( K \) beyond this value has little effect in reducing differential settlement.

A new methodology for the design of buildings in compressible ground has been formulated based on the Relative Stiffness Factor, the expected total settlement, and the limit deformation criterion. In this method of design, the differential settlements are computed.
and used directly to check whether the structure would show distress or not. This enables a designer to either increase the stiffness of the structure, or to select alternate methods such as provision of a surface fill, or ground improvement, or provision of deep foundations.

Experimental evidence is provided to show that masonry foundations undergo total settlements as much as R.C. foundations. It is postulated that whereas the R.C. foundations carry load by bending, masonry foundations carry load by arch action, and hence any comparison between the two types of foundations on the basis of bending would be fallacious. Thus masonry foundations are shown to be suitable for houses built in compressible ground when strip foundations are found to be feasible.

The Vierendeel foundation is a good method of adding stiffness to a structure, and its contribution is significant mainly in framed structures. Again, in the case of framed structures, from considerations both of stiffness and the ability to resist differential settlements, individual pad footings with the columns connected by plinth beams are preferred to individual pad footings.

Lateritic fills lying above compressible layers at the surface are shown to be very effective in reducing the foundation stresses coming on the compressible layer. The fill load also produces settlements in the compressible layer, and these must be allowed for before the commencement of construction. A case study is taken to show that although the settlements due to the structural loads may be small, nevertheless distress can occur in the building because of excessive settlements due to the fill load.

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10.0 References
