STRUCTURAL STIFFNESS AND LIMITING DEFORMATION CRITERIA AS DETERMINED FROM BUILDINGS IN THE LOW LYING AREAS OF COLOMBO

by

Prof. B.L. Tennekoon, Dr. N. Sivakugan and K.T.R. Lakshman

Abstract

At present, the design of foundations is based on limiting total settlements, although it is the differential settlements which cause distress to a building. The differential settelements can be controlled by providing adequate stiffness to a structure.

The settlement patterns of a large number of new buildings in the low lying areas in and around Colombo have been studied. From these results, limiting deformation criteria (based on differential settlement) have been established in terms of angular distortion for framed structures, and deflection ratio for load bearing wall structures.

A non-dimensional parameter called the Relative Stiffness Factor is defined. This takes into account both the stiffness of the structure and the stiffness of the ground. Methods are established for determining the relative stiffness factor. The theoretically determined stiffness are then compared with the actual stiffness of several buildings as observed by the settlement patterns of the buildings, and good agreement is obtained. Hence guidelines are established for increasing the stiffness of the structure.

1.0 Introduction

At present, structural engineers design foundations on the basis of an allowable bearing capacity which depends on

i. adequate safety against shear failure of the soil, and

ii. limiting the total settlement of a building.

Although total settlements do affect the service lines and the aesthetics of a building, it is the differential settlements which cause distress to a building. However, because of difficulties encountered in the estimation of differential settlements and the incorporation of these into design, these differential settlements are generally not used in design. It is simply assumed that the differential settlements that a structure can tolerate can be kept to acceptable values by limiting the total settlement. For example, LEONARDS (1987) recommends limiting the total settlement to 50 mm in sands and 75 mm in clays. However, when designing and constructing buildings in compressible ground, total settlements in excess of these limiting total settlements may be obtained. In such cases, the buildings may still be

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constructed safely provided that the differential settlements are kept within allowable limits by providing adequate stiffness to the structure. This paper gives a method of estimating the overall of the structure, and establishes the limiting deformation criteria, both of which are necessary for the design of structures when the differential settlement have to be directly considered.

2.0 Limiting Deformation Criteria

Two types of limiting deformation criteria are generally used: (i) limiting deformation criteria to prevent 'architectural damage'; i.e. for the onset of visible cracking in plaster. (ii) limiting deformation criteria to prevent 'structural damage'; i.e. for the stability of the structure.

In this paper, the study is restricted to the determination of limiting deformation criteria for the onset of visible cracking.

Studies by MEYERHOF (1956) show that load bearing wall structures have a different mode of deformation from framed structures, and consequently different deformation criteria are recommended for the two types of structures. In the case of load bearing wall structures, the limiting deformation criterion is determined in terms of the deflection ratio (Δ/L) as shown in Fig. 1a; and in framed structures the limiting deformation criterion is obtained in terms of the angular distortion (β) as shown in Fig. 1b.

2.1 Limiting deformation criterion for load bearing wall structures.

BURLAND and WORTH (1975) have from theoretical considerations shown that the limiting deflection ratio for load bearing wall atructures will vary depending on the brittleness of the building material, the length to height ratio, the relative stiffness in shear and bending, and the mode of deformation (sagging or hogging). However, in this study, as a first approximation, all load bearing wall structures are taken together in one category for the determination of the limiting deflection ratio.

The limiting deflection criterion has to be studied by making use of the settlement measurements that have been made on the buildings that are being monitored, together with an assessment of whether the building is free of cracks or not. The buildings that have been analysed are given in Appendix I.

Taking a typical example, the plan view of Block D of the Nawagampura Housing Scheme is shown in Fig. 2a. Settlement markers have been placed at the points numbered 1 to 10 in the figure. The settlement markers consist of durable ceramic tiles (50mm x 50mm) which are fixed on flat horizontal surfaces such as the DPC. Settlement measurements were taken using a precision **levelling** instrument with reference to a carefully established benchmark. Benchmarks were installed in structures which are considered to be stable or undergoing negligible settlements over the period of monitoring. Several benchmarks were installed within a particular area and these were tied to each other to confirm their stability. It has been assessed that the settlement measurements could be obtained to an accuracy of 1mm. Typical values of total settlement and differential settlement (after correcting for rigid body displacements) are shown in Fig. 2b.

Considering the wall along 8-9-10-1, the deflection ratio is computed as (1/1725).

For each building that is being monitored, the deflection ratio is determined on each date that the settlements have been measured. The relationships between deflection ratio and average settlement for the different buildings are shown in Figs. 3a and 3b. From these results, it is estimated that cracking in load bearing wall structures

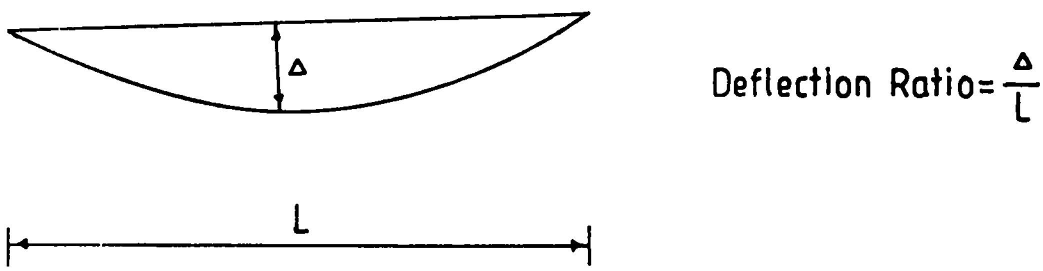


Fig. 1a - Definition of Deflection Ratio for load bearing wall structures

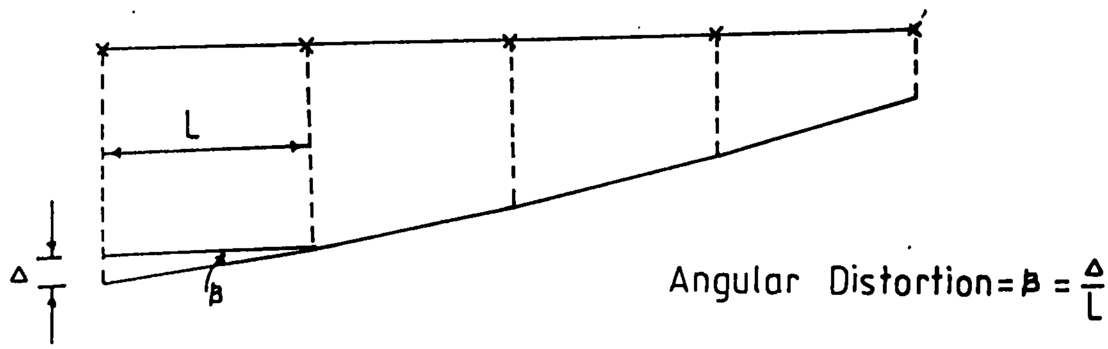


Fig. 1b - Definition of Angular Distortion for framed structures

Nawagampura Block D

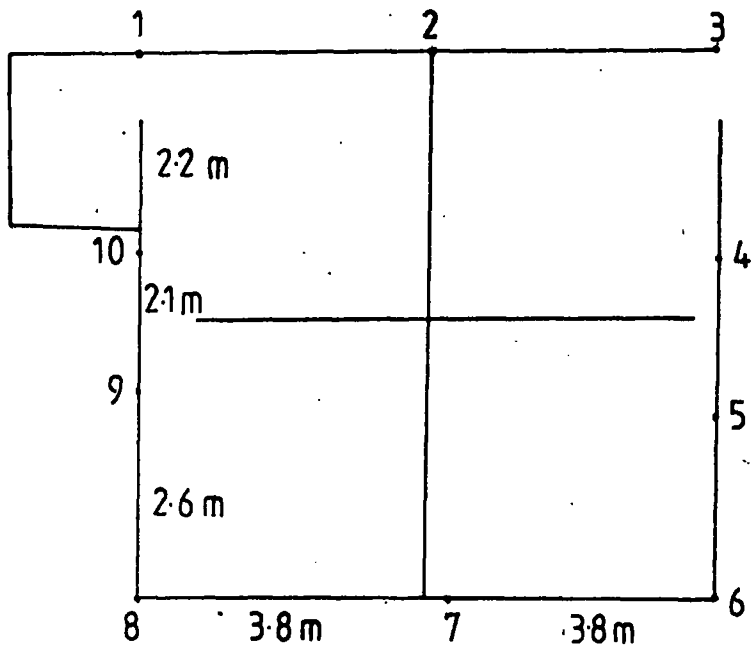


Fig. 2a - Plan view of Building

Settlement as at 88-03-04

Point No.	1	2	3	4	5	6	7	8	9	10
Settlement (mm)	200	217	230	-	199	182	169	153	168	181
Δ (mm)	0	2.5	1	-	5	0	1.5	0	2.7	4

$\rho_{average} = 191\ mm$

Along 8-9-10-1 $\Delta = 4$ mm $L = 6.9$ m

$$\frac{\Delta}{L} = \frac{1}{1725}$$

Fig. 2b - Computation of Deflection Ratio

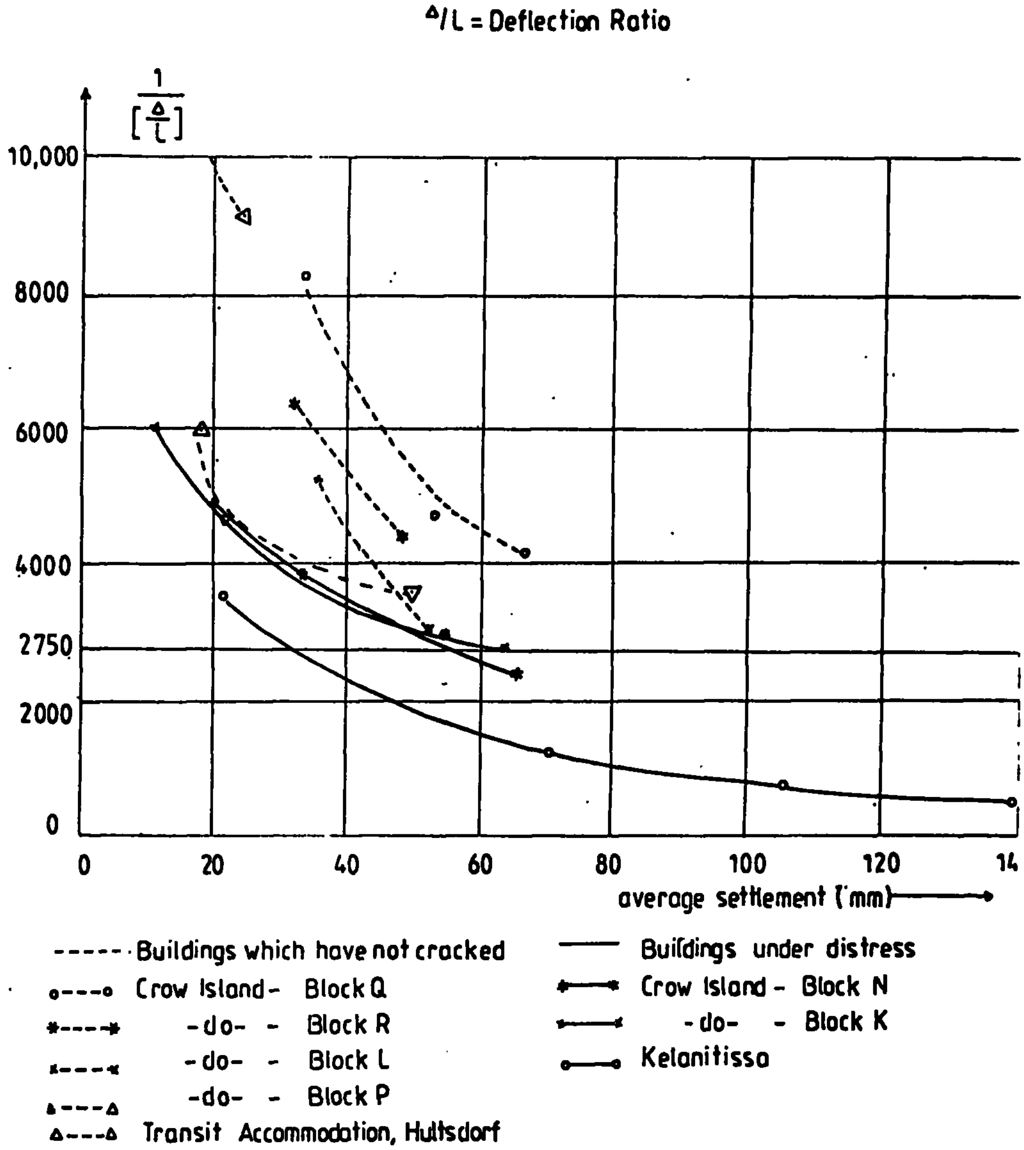


Fig. 3a Relationship between deflection ratio & average settlement for load bearing wall structures

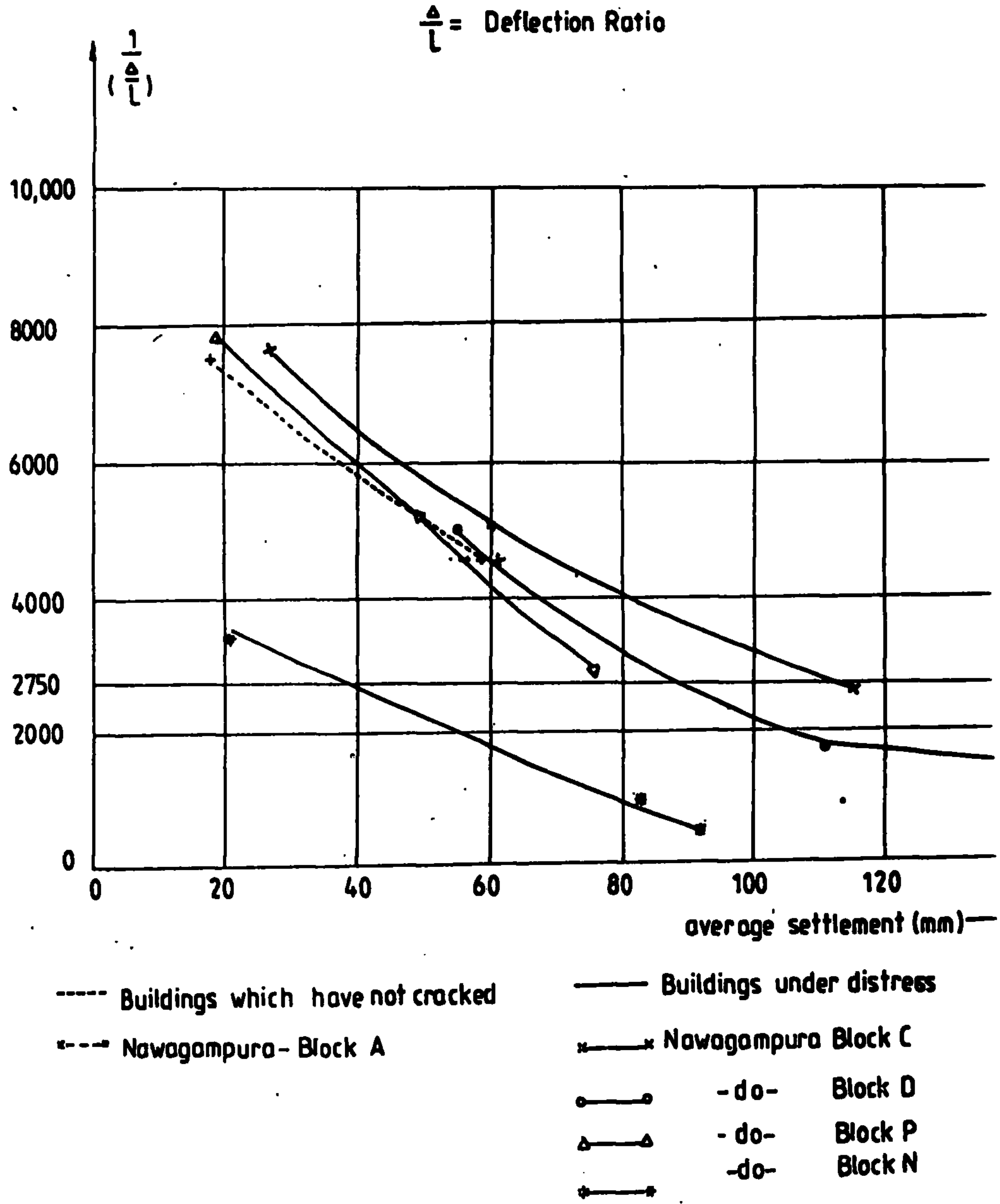
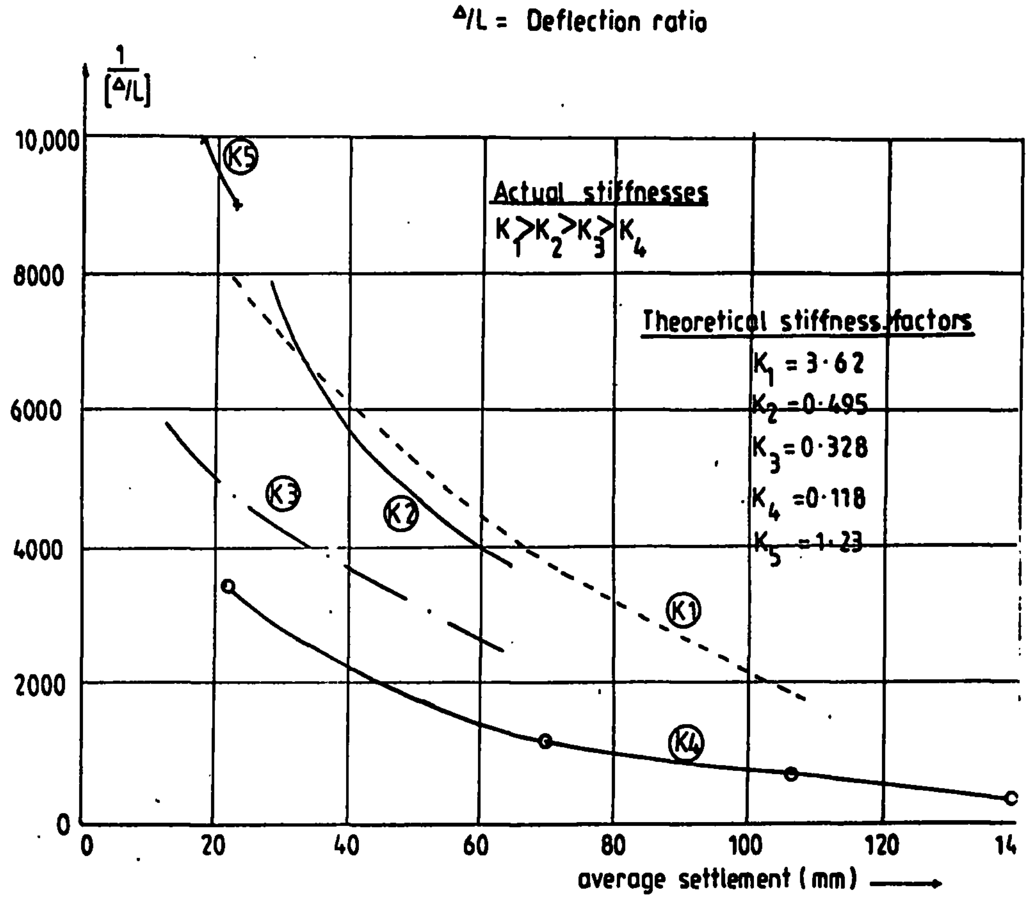


Fig. 3b Relationship between deflection ratio & average settlement for load bearing wall structures



----- Nawagampura Blocks D, P, N
——— Crow Island Blocks Q, R (veirendeel Foundation)
—·— Crow Island Blocks P, N, K (inverted T Foundation)
o—o Kelanitissa
•—• Transit Accommodation, Hultsdorf

FIG. 3C Relationship between deflection ratio & average settlem for load bearing wall structures commence at a deflection ratio of (1/2750).

For the purposes of comparision, the limiting values given by MEYERHOF (1956) and POLSHIN and TOKAR (1957) are: (i) MEYERHOF gives a value of (1/2500). (ii) Polshin and Tokar give values of

a. $(1/3500) < (\Delta/L) < (1/2500)$ for $(L/H) < 3$
b. $(1/2000) < (\Delta/L) < (1/1500)$ for $(L/H) > 5$

2.2 Limiting deformation criterion for framed structures

The framed structures were studied similarly, and the buildings that have been analysed are given in Appendix I.

Taking typical example, the plan of the IDB building at Peliyagoda is shown in Fig. 4a. Settlement markers have been placed at the column points numbered 1 to 12 in the figure. The settlement markers consist of steel hooks with a rounded edge installed in the columns. The settlement measurements were made as described earlier. Typical values of total settlement and differential settlement are shown in Fig. 4b.

Considering the columns along 1-2-3-4-5, the maximum angular distortion is computed as (1/258). For each building that is being monitored, the maximum angular distortion is determined on each date that the settlements have been measured. The relationship between maximum angular distortion and average settlement for the different buildings are shown in Fig. 5. From these results, it is estimated that visible cracking in framed structures commence at an angular ratio of (1/300).

SKEMPTON AND MACDONALD (1956), GRANT ET. AL. (1974), BURLAND AND WORTH (1975) all report a limiting angular distortion of 1/300 for framed structures.

3.0 Stiffness of the structure

It is well known that differential settlements can be controlled by providing adequate stiffness to a structure. The stiffness of the structure refers not only to the stiffness of the foundation but also to the stiffness of the superstructure. But as stated in ANON (1977), the overall stiffness of a structure is difficult to assess with any accuracy because in practice the degree of fixity at joints is uncertain; cladding and in-fill panels have varying degrees of fit; the geometry of the structure at any time during construction has a significant influence on stiffness and the distribution of forces; etc. Nevertheless, rational design of structures where settlements are high can be done only when it is possible to quantify the stiffness.

In the case of framed structures, MEYERHOF (1953) proposed that the stiffness of the structure can be approximated to the sum of two components:

i. the stiffness due to the infilling $= \frac{E_w I_w b^2}{2H^2}$

where E_w, I_w are the Young's Modulus and moment of inertia respectively of the wall;

b is the length of the building in the direction along which bending is considered; and

H is the total height of infilling.

ii. the stiffness due to the frame $= \sum E_b I_b \left(1 + \frac{K_l + K_u}{K_b + K_l + K_u}\right) b^2$

where E_b I_b are Young's Modulus and moment of inertia respectively of the beam (or beam and slab);

K_l, K_u, K_b are the average stiffness of the lower column, upper column, and beam respectively;

l is the length of a bay in the framed structure in the direction along which bending is considered;

IDB, Peliyagoda

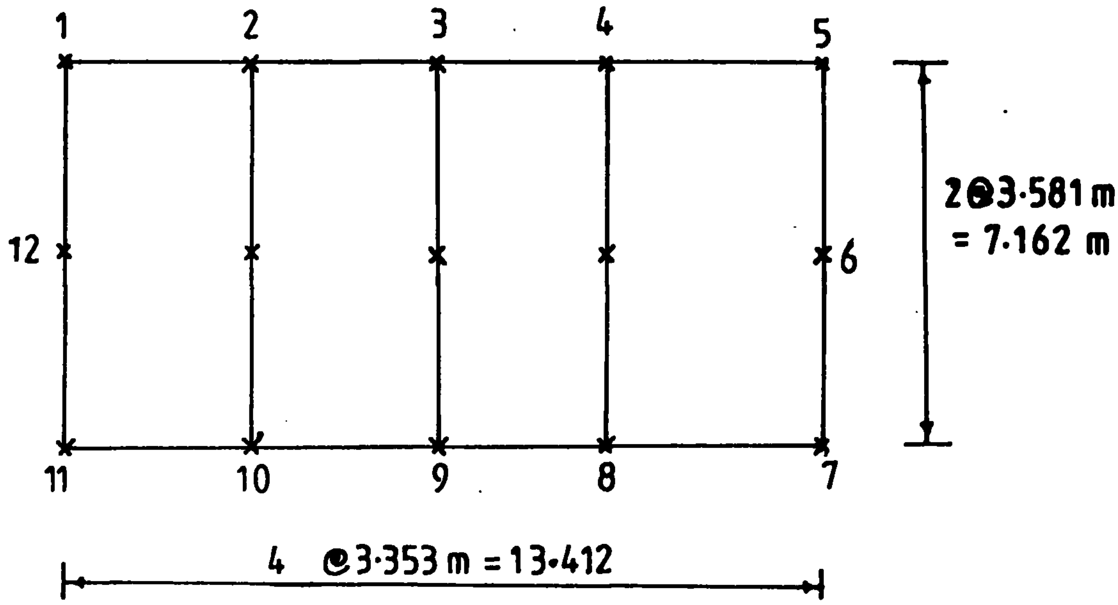


Fig.4a - Plan view of Building

Settlements as at 88-03-18

Point No.	1	2	3	4	5	11	12	1
Settlement (mm)	152	149	145	132	125	138	149	152
Δ (mm)	3	4	13	7		11	3	

Along 1-2-3-4-5 $\quad \Delta_{max} = 13\ mm$

$$\beta_{max} = \frac{13}{3{\cdot}353 \times 1000} = \frac{1}{258}$$

Fig.4b - Computation of maximum angular distortion

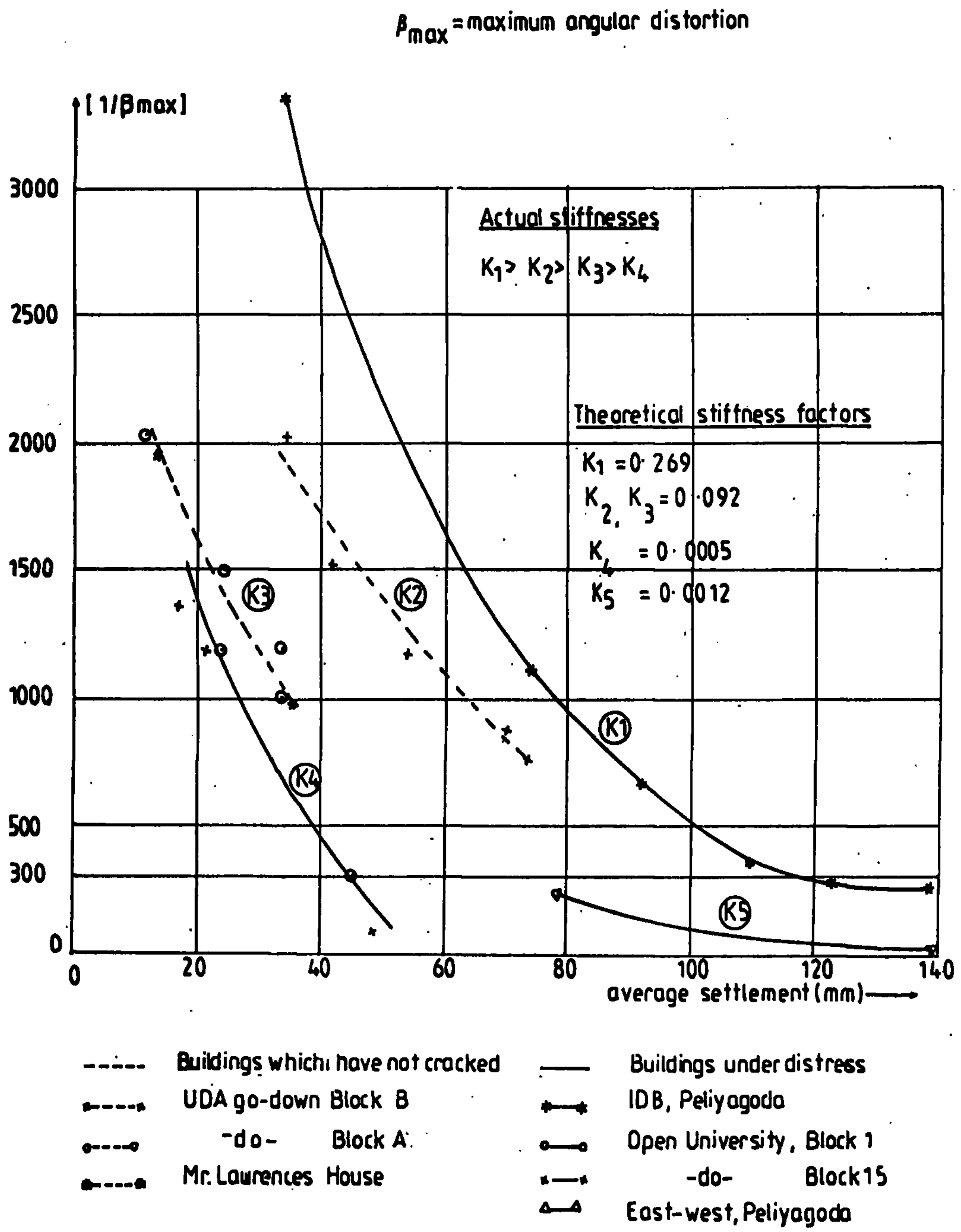


Fig 5 Relationship between angular distortion & average settlement for Framed structures

and the summation is to be carried out for each floor.

This method was endorsed by the American Concrete Institute, DE SIMONE (1966); and it has also been adopted by the Indian Standards Institution, ANON (1972).

In the case of load bearing wall structures, there is no recommendation as to the method of computing the stiffness of the structure. Therefore, as a first approximation, the stiffness of the structure is computed in the same manner that the stiffness of the infilling is determined for framed structures.

The stiffness of the structures as theoretically determined are shown in Tables la and lb. It is seen from Table la that the contribution of the infill or cladding to the structure stiffness is very large in comparison to the contribution from the frame. It is reported in ANON (1977) that

i. the contribution that the infill panels in framed structures makes towards the overall stiffness of the structure is dependent on the tightness of fit in the frame.

ii. when there are openings in the infill panels, stiffness will be reduced. e.g. it is recommended that when the size of openings exceed 1/6 of the frame opening, the stiffness from the infill panels should be ignored.

Thus, in many domestic buildings of framed structures with a large amount of openings for doors and windows, the contribution from the walls is negligible. However, considerable stiffening can be provided beneath the ground by providing Vierendeel type foundation system. As an example of this, of the structures analysed, in the case of Mr. Lawrence's house almost all the stiffness of the structure has been contributed by the Vierendeel foundation.

In the case of load bearing wall structures, design from strength criteria require that there be a greater provision of walls than in a framed strucutre. Thus, often load bearing wall structures tend to be stiffer than framed structures, and consequently the differential settlements associated with the load bearing wall structures are smaller, although they may still lead to cracking because the limiting deformation criterion allows a smaller defferential settlement.

4.0 Relative Stiffness of Structures

Just as much as very rigid structures undergo little differential settlements, structures on very weak soils could "sink" as a rigid body. Therefore, MEYERHOF (1953) defined a term called the Relative Stiffness Factor (K) as

$$K = \frac{\text{Stiffness of structure}}{\text{Stiffness of soil}}$$

It is this parameters K which can be used as a measure to evaluate differential settlements.

MEYERHOF (1953) has derived theoretically a relationship between $\left(\frac{\Delta\, max}{\rho}\right)$ and K for a uniformly loaded rectangular foundation of width B and length L. This relationship is shown in Fig. 6, and it shows that if K can be made to be greater than about 0.5, the foundation tends to be extremely rigid with $\left(\frac{\Delta\, max}{\rho}\right) \simeq 0.08$. ($\Delta$ max is the maximum differential settlement, and ρ is the average settlement).

For the stiffness of the soil, MEYERHOF (1953) has obtained

$$\text{Stiffness of soil} = E_s b^3 a$$

where E_s is the modulus of elasticity of the soil;

b is the length of the building in the direction of bending;

and a is the length perpendicular to the section under investigation.

This method has also been adopted by the Indian Standards Institution; ANON (1972).

The method of determining the stiffness of the soil now gets modified to the determination of the modulus of elasticity of the soil, and the method of determination of the latter would depend on the type of soil.

The types of compressible soils usually encountered in the low lying areas are :

i. loose deposits of sand;
ii. soft clays - this would include inorganic clays, organic clays, and peaty soils;
iii. lateritic fill overlying a weak soil layer, the latter being compressible.

4.1 Loose deposits of sand

The SPT value is often too small to be used for the computation of E_s. A more suitable test is the Static Cone Test in which MEYERHOF (1965) has recommended the use of the empirical relationship $E_s = 1.9\ q_c$ where q_c is the static cone resistance.

In the absence of static cone resistance results, the recommendations of the Indian Standards Institution - ANON (1972) - have been used. According to this recommendation, for loose sand

E_s = 200 to 500 kg/cm²

i.e. E_s = 20,000 to 50,000 kN/m²

4.2 Soft clays

On the basis of the analysis of observations of shallow foundations on clay, MEYERHOF (1953) recommends that for soft cohesive soils E_s = (25 to 80) x cohesion, when consolidation settlements are also considered.

Soft clays, especially the peaty soils, are very difficult to sample without causing disturbance to the soil structure. However, a limited number of tests have been carried out in the peaty soil at the UDA Go-downs, Orugodawatte. These tests gave

$c_u \simeq 0.35$ kgf/cm²

Taking $E_s = 50 \times c_u$,

E_s for the peaty soil works out as 1700 kN/m²

4.3 Lateritic fill overlying a weak soil layer

FRASER AND WARDLE (1976) have proposed an approximate graphical method for determining the equivalent E_s for a multi-layered soil system. According to this method, the elastic parameters of each layer is weighted according to its influence on settlements. A worked example of this method is given in Appendix II for the UDA Go-downs, Orugodawatte.

In using this method it is necessary to determine the E_s values for the different individual soil types. For lateritic soils one of the following methods may be used.

i. the recommendation of the Indian Standards Institution - ANON (1972) - that E_s be taken as the second modulus of the load settlement curve from an unconfined compression test corresponding to 50% of the unconfined strength.
ii. the recommendation of DOWRICK (1977) given in Table 2. For this analysis, E_s for compacted laterite has been taken as 50,000 kN/m².

4.4 Comparison of theoretical and experimental stiffness of buildings

The stiffness of the soil for the cases analysed are given in Tables 1a and 1b.

The relative stiffness factors for the cases analysed can now be computed, and these are also given in Tables 1a and 1b.

As mentioned earlier, the assessment of the relative stiffness factor is based on certain idealisations which may be in error. A better indication of the relative stiffness of structures can be obtained by analysing the results of the graphs of,

i. angular distortion versus settlement, for framed structures;

ii. deflection ratio versus settlement for load bearing wall structures.

The larger the relative stiffness of the structure, the smaller would be the differential settlement for any given total settlement.

The relationship between angular distortion and settlement for framed structures is shown in Fig. 5. It is observed that the building with the highest relative stiffness factor is the IDB building at Peliyagoda. This is mainly because the foundations have been placed just above the peat layer and hence the stiffness of the ground is low. However, it should be noted that although buildings constructed just above weak soils tend to be stiffer than if the same building was placed after placing a compacted fill, the former also tends to settle considerably more than the latter. An example of the latter is the UDA Go-downs at Orugodawatte where the foundations are placed on the compacted fill at an elevation much higher than the peat layer. By this process the stiffness of the ground has increased and consequently there is a reduction in the relative stiffness factor. But the lateritic fill has been very effective in reducing the pressure coming on the weak layer and thus reducing the total settlement.

For comparison, the IDB building at Peliyagoda had been designed for a bearing pressure of 40 kN/m^2 and had undergone a settlement of (125-152 mm); whereas the UDA Go-downs had been designed for a bearing pressure of 160 kN/m^2 and had undergone a settlement of (60-87 mm).

This is despite the fact that the thicknesses of the two peat layers are 1.98 m for the IDB building site, and 5.90 m for the UDA Go-downs site.

Again from Fig. 5, it is observed that the East-West Building at Peliyagoda has a very small relative stiffness factor, and it has also undergone very large settlements. In this case, although the stiffness of the soil is relatively small (because the foundations are placed near the peat layer), the relative stiffness factor is extremely small because

i. the building has large dimensions in plan; and

ii. the structure stiffness is small because the building is a framed structure in which any contribution to stiffness of structure from the walls had been neglected because of the large amount of openings.

Another observation from Fig. 5 is that although Blocks A and B of the UDA Go-downs, Orugodawatte are structurally similar, they show different settlement behaviour - Block B has a higher relative stiffness than Block A. It is concluded that this difference is most probably

Project	Stiffness of structure (EI) walls (kN.m²)	(EI) frame (kN.m²)	(EI) total (kN.m²)	Stiffness of soil E_s (kN/m²)	a (m)	b (m)	(EI) soil (kN.m²)	Relative Stiffness Factor (K)
1. IDB, Peliyagoda	7.7×10^6	0.1×10^6	7.8×10^6	1700	7.16	13.41	29×10^6	0.269
2. UDA Go-downs	265.4×10^6	0.1×10^6	265.5×10^6	5130	19.8	30.48	2876×10^6	0.092
3. Mr.Lawrence's house	8.7×10^6	0.1×10^6	8.8×10^6	10000 to 20000	7.54	9.98	75×10^6 to 150×10^6	0.118 to 0.059
4. East-West, Peliyagoda	7.8×10^6	0.1×10^5	7.3×10^6	2000	35	45	6380×10^6	0.0012
5. Open University, Nawala. Block 1.	113.9×10^6	0.1×10^6	114×10^6	5000	12	45	5467×10^6	0.021

Table 1a - Stiffness determination for Framed Structures

Project	Stiffness of structure	Stiffness of soil				Relative Stiffness Factor (K)
	$(EI)_{structure}$ $(kN.m^2)$	E_s (kN/m^2)	a (m)	b (m)	$(EI)_{soil}$ $(kN.m^2)$	
1. Nawagampura	8.258×10^6	2000	2.6	7.6	2.283×10^6	3.62
2. Crow Island, (Veirendeel)	91.77×10^6	5000	3.81	21.34	185.1×10^6	0.495
3. Crow Island, (Inverted T)	60.76×10^6	5000	3.81	21.34	185.1×10^6	0.328
4. Devco Showa	16.9×10^6	5000	9.0	27.0	885.7×10^6	0.019
5. Transit Accommodation	10.8×10^6	2000	6.0	9.0	8.748×10^6	1.23
6. Kelanitissa, Block A	36.57×10^6	2000	9.3	25	309.4×10^6	0.118

Table 1b - Stiffness determination for Load Bearing Wall Structures

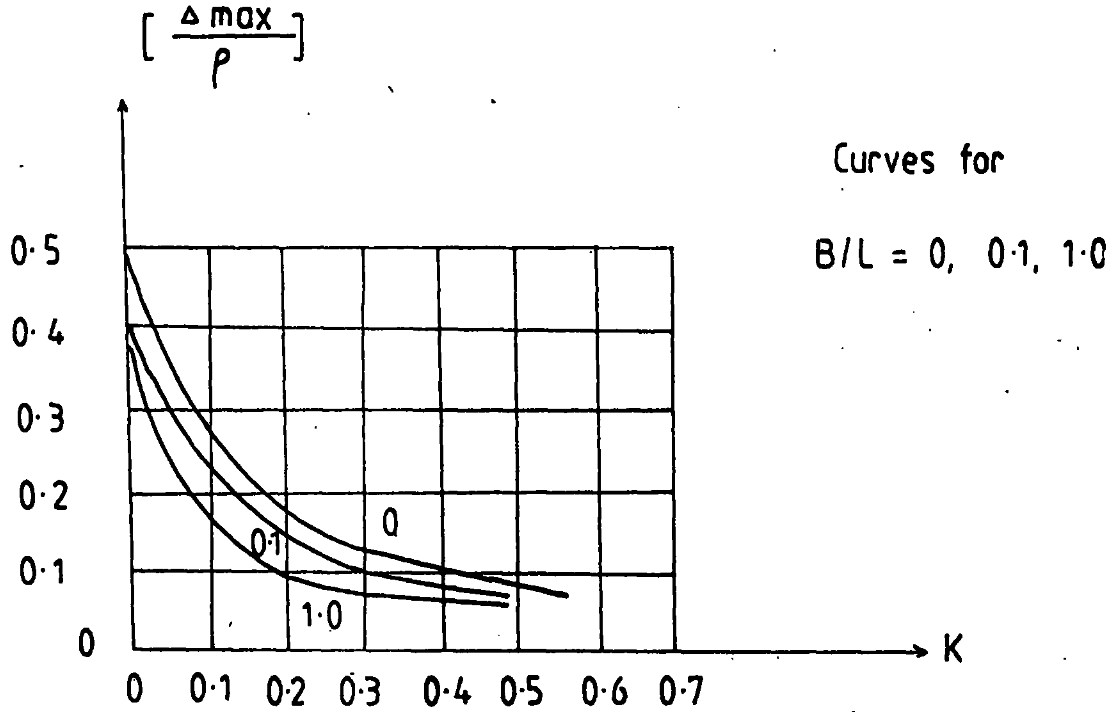


Fig. 6 Effect of Relative stiffness Factor on differential settlement of Uniformly loaded rectangular footing [MEYERHOF, 1953]

Soil Type	E (kN/m^2)
Soft clay	upto 15,000
Firm, stiff clay	10,000 to 50,000
Very stiff, hard clay	25,000 to 200,000
Silty sand	7000 to 70,000
Loose sand	15,000 to 50,000
Dense sand	50,000 to 120,000

Table 2 - Typical E-values for soils

(Reference - DOWRICK (1977), p.139)

due to the variations in the ground conditions at the site.

The relationship between deflection ratio and settlement for load bearing wall structures is shown in Fig. 3c. (This is a combination of Figs. 1b and 1c, but using the average curve of several blocks in the projects where a large number of similar buildings have been monitored).

A comparison of the two curves of the Crow Island Housing Scheme - one curve for the houses with Vierendeel foundations and the other curve for houses with the inverted T-foundations shows that the use of the Vierendeel foundation system has increased the stiffness of the structure.

The Transit Accommodation at Hultsdorf have a theoretically determined Stiffness Factor of 1.23. This high stiffness Factor together with the relatively small total settlements (ρ = 25 mm) has resulted in extremely small differential settlements (Δ = 2 mm), and consequently these structures show no distress.

In the case of the Nawagampura Housing Scheme, the theoretically determined Stiffness Factor is the highest of all the buildings analysed; yet in its performance (as observed from Fig. 3c), it is similar to the Crow Island Housing Scheme with the Vierendeel foundations, which has a much lower Stiffness Factor. A possible reason is that the increase in relative Stiffness Factor beyond 0.5 had only a marginal effect in reducing the differential settlements of a structure. (This would be similar to the results of MEYERHOF (1953) shown in Fig. 6.

5.0 Relationship between Differential Settlement and Structure Stiffness at Failure

In this section, a relationship will be established between (Δ/ρ) at failure and the Relative Stiffness Factor (K). (Δ is the differential settlement as defined earlier and ρ is the average total settlement).

For framed structures, failure is assumed to occur at an angular distortion of 1/300, and then using the result of Fig. 5, the relationship between (Δ/ρ) and K shown in Fig. 7a is obtained.

Similarly, for load bearing wall structures, failure is assured to occur at a deflection ratio of 1/2750, and then using the result of Fig. 3c, the relationship between (Δ/ρ) and K shown in Fig. 7a is obtained.

6.0 Conclusions

It has been established that the limiting deformation criteria for the onset of cracking are,

i. angular distortion of 1/300 for framed structures; and

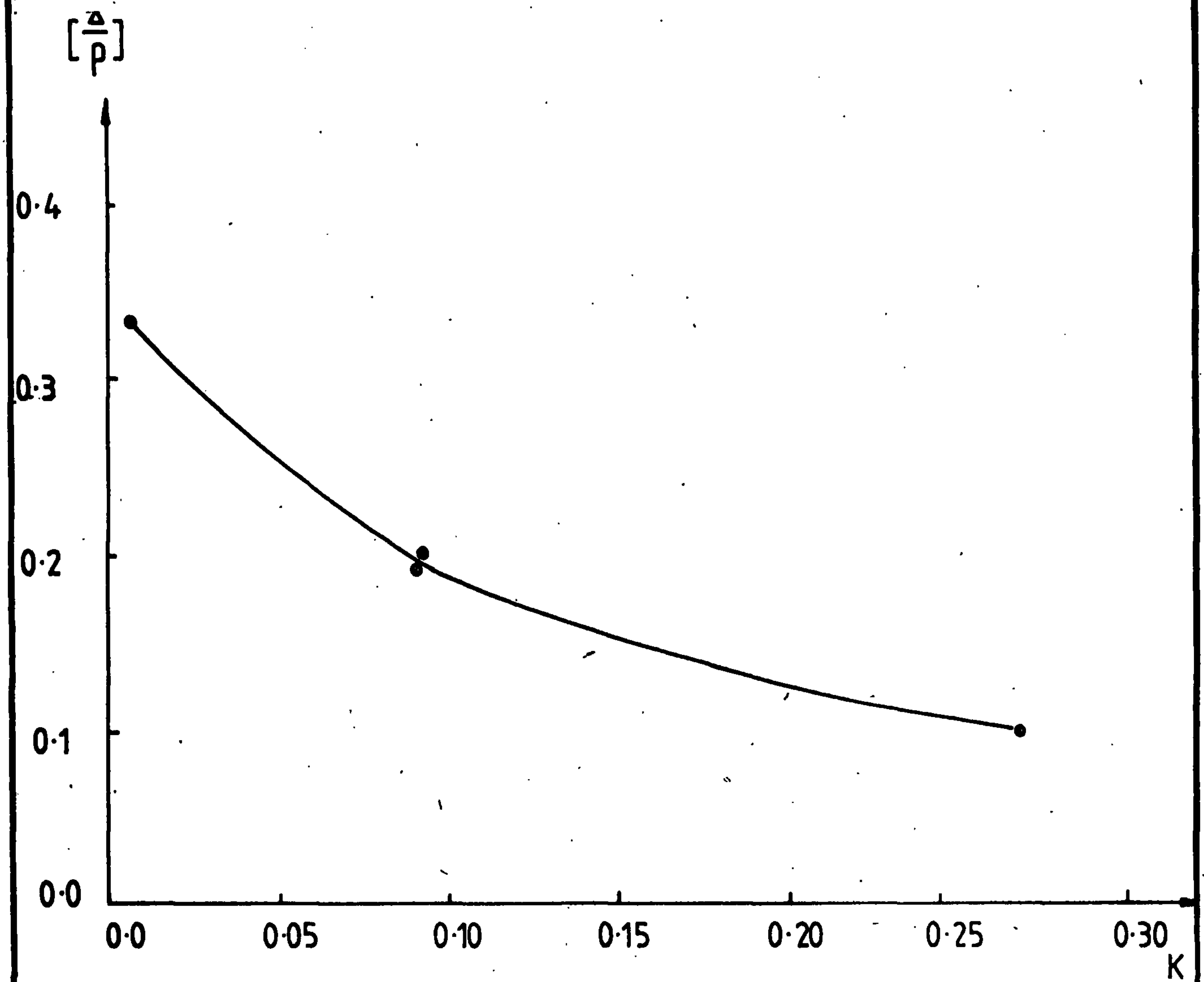
ii. deflection ratio of 1/2750 for load bearing wall structures.

It has also been established that although the overall stiffness of a structure is difficult to assess accurately, reasonably approximate estimates can be made. Reasonably good correlation has been obtained between the actual stiffness as determined by the settlement pattern of the building, and the theoretical stiffness based on definitions for the stiffness of the structure and the stiffness of the soil.

From these it has been established that,

i. for framed structures, the contribution to structure stiffness from the infilling is much larger than the contribution from the frame.

ii. the presence of openings for

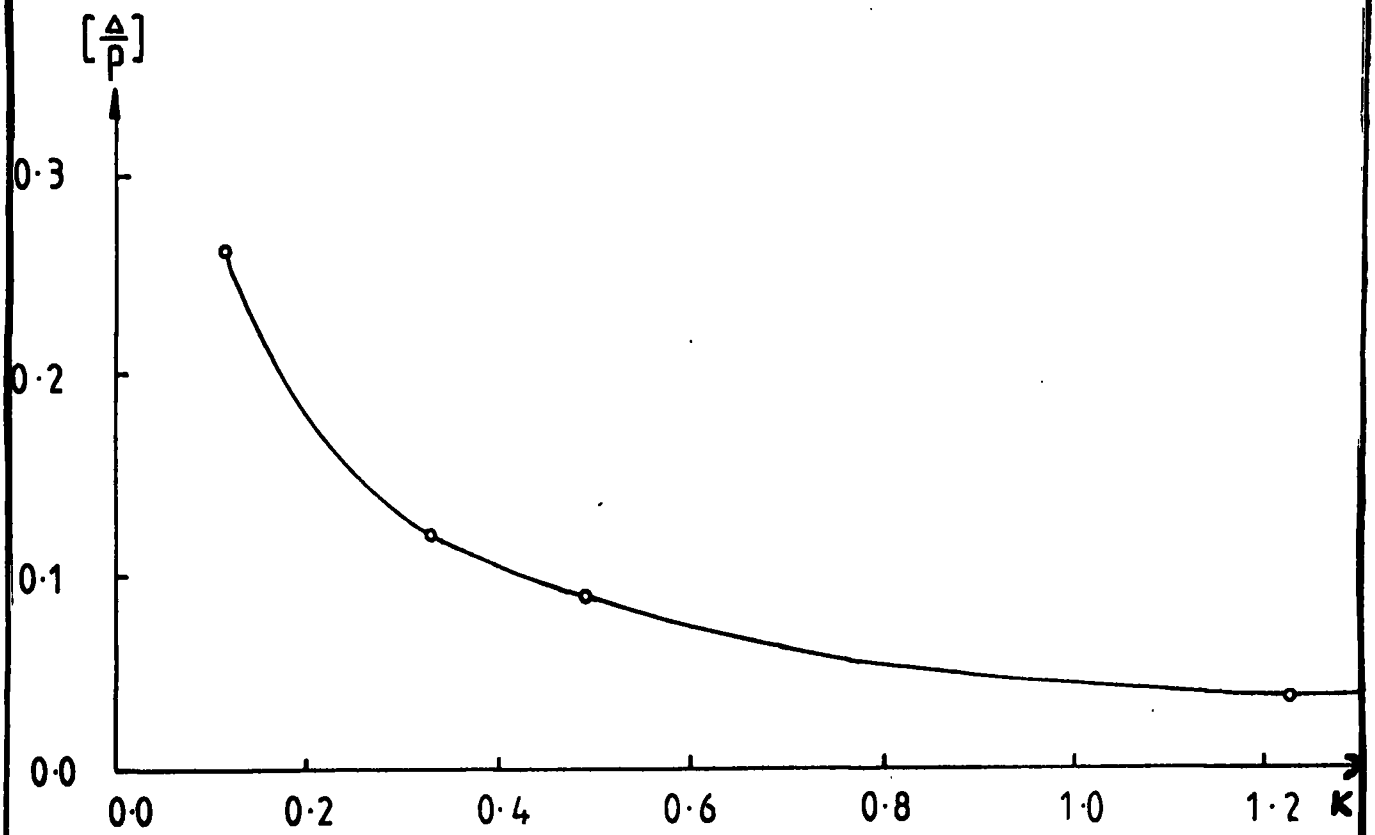


K = Theoretically determined Relative stiffness Factor

Δ = Maximum differential settlement between columns at failure, assuming failure at $\frac{\Delta}{L} = \frac{1}{300}$

p = Average settlement corresponding to failure at $\frac{\Delta}{L} = \frac{1}{300}$

<u>Fig. 7a -</u> <u>Relationship between $[\frac{\Delta}{p}]$ and K at failure, assumed to occur at $\frac{\Delta}{L} = \frac{1}{300}$, for framed structures</u>



K = Theoretically determined Relative stiffness Factor

Δ = Maximum differential settlement at failure, assuming

failure at $\frac{\Delta}{L} = \frac{1}{2750}$

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

Fig. 7b - Relationship between [$\frac{\Delta}{p}$] and K at failure, assumed to occur at $\frac{\Delta}{L} = \frac{1}{2750}$, for load bearing wall structures

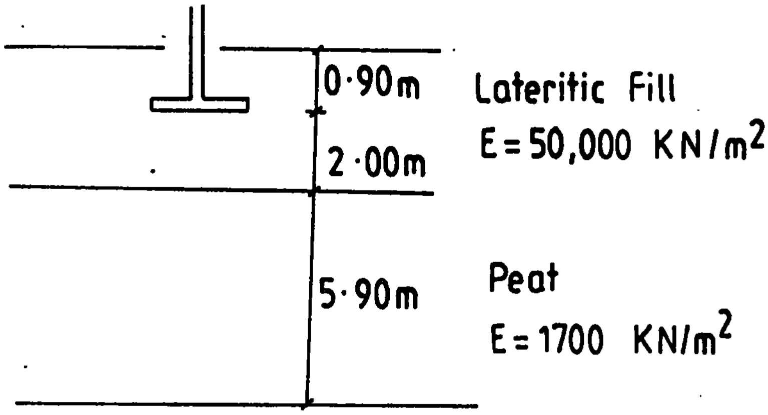


Fig. 8a - Ground conditions at UDA Go-downs, Orugodawatte

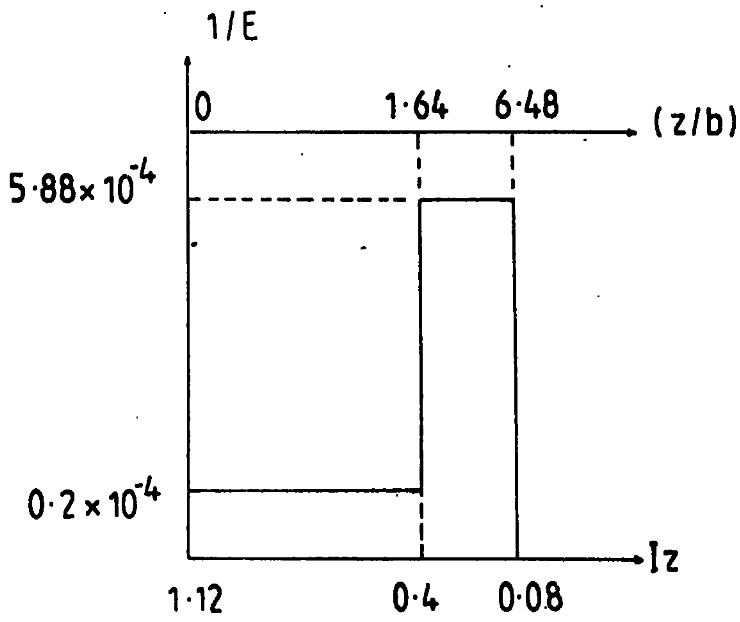


Fig. 8 b - Relationship between 1/E & Iz using results of Fraser & Wardle (1976) doors and windows in the walls reduce considerably the stiffness of the structure.

iii. load bearing wall structures tend to be stiffer than framed structures.

iv. the use of the Vierendeel girder foundation system increases the stiffness of the structure.

v. variations of relative stiffness for similar buildings can be obtained because of variations in the ground conditions.

vi. buildings with smaller plan dimensions are stiffer than buildings with larger dimensions.

vii. buildings constructed on weak soils tend to be stiffer than those constructed at the same site after filling with a hard fill. However, in the former case, they also show considerably more settlement.

viii. the increase of the relative stiffness factor beyond a value of about 0.5 has little effect on the differential settlements.

Relationship have also been obtained between (Δ/ρ) at failure and the relative stiffness factor (K) for (a) framed structures, and (b) load bearing wall structures.

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APPENDIX I

LIST OF STRUCTURES WHICH HAVE BEEN STUDIED

Framed Structures

1. I.D.B., Peliyagoda
2. U.D.A. Go-downs, Orugodawatte - Block A
3. - do - - Block B
4. Mr. Lawrence's house, Rajagiriya
5. Open University, Nawala - Block 1
6. - do - - Block 15
7. East-West, Peliyagoda

Structures with Load Bearing Walls

1. Nawagampura Housing Scheme - Block C
2. - do - - Block D
3. - do - - Block A
4. - do - - Block N
5. - do - - Block P
6. Crow Island Housing Scheme - Block P
7. - do - - Block L
8. - do - - Block N
9. - do - - Block K
10. - do - - Block Q
11. - do - - Block R
12. *Devco Showa, Peliyagoda
13. Transit Accommodation, Hultsdorf - Block A
14. - do - - Block B
15. Kelanitissa Power Station Premises, Wellampitiya - Block A

* Settlement monitoring of this structure commenced three years after the completion of this building. Most of the primary consolidation was over, and hence the total settlements cannot be determined.

APPENDIX II

DETERMINATION OF THE EQUIVALENT E_s FOR A MULTI-LAYERED SOIL USING THE METHOD OF FRASER AND WARDLE (1976)

Consider the example of the site at the UDA Go-downs, Orugodawatta.

Footings of dimensions 1.219 m x 1.219 m have been placed at a depth of 0.90 m as shown in Fig. 7a; (i.e. b = 1.219)

The compressible peat layer is 5.90 m thick, and a compacted lateritic fill of thickness 2.0 m has been placed above it.

The E values for the lateritic fill and the peat have been estimated as 50,000 kN/m² and 1700 kN/m² respectively.

Therefore, in the range $0 < z < 2.0$

i.e. $0 < (z/b) < 1.64$, $\frac{1}{E} = \frac{1}{50,000} = 0.2 \times 10^{-4}$

Similarly in the range $2.0 < z < 7.90$

i.e. $1.64 < (z/b) < 6.48$, $\frac{1}{E} = \frac{1}{1700} = 5.88 \times 10^{-4}$

From Fig. 17 of Fraser and Wardle (1976), the following influence coefficients were obtained:

i. $(z/b) = 0$, $I_z = 1.12$

ii. $(z/b) = 1.64$, $I_z = 0.4$

iii. $(z/b) = 6.48$, $I_z = 0.08$

Using these values, Fig. 7b has been constructed.

The equivalent E_s of the multi-layered soil ($\bar{E}$) is determined

$$\frac{1}{\bar{E}} (\Delta I_z) \text{ total} = \sum \frac{1}{E_i} (\Delta I_z)_i$$

This gives E = 5130 kN/m²