PROCEEDINGS

OF THE

SEMINAR ON

APPROPRIATE FOUNDATIONS FOR CONSTRUCTION IN LOW LYING MARSHY AREAS

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P R E F A C E

With most of the highlands in the urban areas heavily built up, the attention of the planners and developers has been focussed on the development of low lying marshy areas in and around the Colombo city. The extremely weak, highly compressible subsoils prevailing in the marshy areas pose many a problems to the developers.

The National Building Research Organisation (NBRO) of the Ministry of Policy Planning and Implementation (then under the Ministry of Local Government, Housing and Construction) realising the importance of developing suitable guidelines for the development and construction activities in the low lying areas launched several research projects with the assistance of United Nations Development Programme.

NBRO, considering it appropriate to disseminate the findings so far made, and to exchange opinions with the personnel involved in the development of low lying areas, decided to hold a one day Seminar.

Many individuals and agencies, both in the public sector and private sector, involved in the building and construction industries cooperated with NBRO enthusiastically to make the projects successful. We take this opportunity to thank them for their suggestions, contributions and assistance.

We wish to keep on record with appreciation the efforts of many scientific and technical personnel who were associated with the projects from their inception.

We also thank the staff at NBRO without whose efforts this Seminar would not have been possible.

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Organising Secretaries
Seminar on Appropriate Foundations
for Construction in Low Lying Areas

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UNDP & UNCHS ASSIST GOVERNMENT OF SRI LANKA TO REDUCE COST OF CONSTRUCTION ON LOW-LYING MARSHY LANDS

KALYAN RAY, UNCHS ENGINEERING ADVISOR

Over the years the City of Colombo has grown in the North-South direction along the costal belt skirting the vast expanse of low-lying marshy lands to the east. Till the mid-seventies, low-lying marshy areas were generally regarded by developers as unsuitable and unworthy of development due to the high cost of reclamation and subsequent construction, often requiring deep foundations.

The situation, however, drastically changed following the liberalisation of Government policies in 1977. The accelerated pace of development imposed severe strain on available lands particularly in and around the City of Colombo. By 1978, when the Colombo Master Plan was produced, the availability of buildable land had dwindled to less than 10% of the total city area and most of these lands were situated in low-lying marshy lands requiring reclamation.

By the early eighties, much haphazard development was taking place in these low-lying areas both by private as well as public sector agencies. Largely due to lack of information, as to the nature and engineering behaviour of the sub-soil in marshy areas, most of these constructions were taking place using conservative design practices, providing deep or heavy foundations, adding substantially to the cost of construction. This high cost of construction made it almost impossible for the low-income groups to build houses in these areas. On the other hand, many projects were executed in these areas in a rather casual manner without providing necessary foundation to withstand large order of settlement that the structures would undergo when built on these lands. Thus, many buildings developed foundation and structural problems very soon after construction, requiring very costly treatment for their restoration.

In 1981, the soil testing laboratory, set up in the Urban Development Authority with UNDP/UNCHS technical assistance, undertook for the first time, a systematic investigation of engineering behaviour of sub-surface soils in the marshy areas. This study, which was carried out by a small team of engineers under the guidance of the present Engineering Adviser, particularly examined the low-lying areas in Peliyagoda, Attidiya, Orugodawatta and Bloemendhal. All these areas were earmarked for early development by UDA, and the types of structures likely to be constructed were generally known.

As a part of this study, field investigation and laboratory analysis of soil samples were carried out to develop a set of characteristic soil profiles based on the number, sequence and extent of various horizons and also the variation of physical and engineering properties of each horizon. These characteristic profiles were then used to formulate a set of soil series having similar morphological and genetic factors. The main soil series, identified in this study was designated as Muthurajawela series. The study also evaluated the engineering response of each soil series to various civil engineering activities likely to be undertaken in these areas. A preliminary attempt was also made in this study to identify suitable foundation types under different sub-surface conditions. Suitability of ground treatment was also examined for each soil series.

The limited scope of this study, and preoccupation of the soil testing laboratory at this time with important Government projects, did not permit a more elaborate and in-depth examination of foundation and superstructure response to different ground conditions, and particularly, soil-structure interaction, information on which was crucial for optimum design of both foundation and super-structure. However, this study provided a sound basis for future research on development of this area.

Following the establishment of the National Building Research Organisation (NBRO) in 1984, the Government again requested UNDP for additional funds to strengthen the existing soil testing laboratory and to organise a comprehensive research project for development of appropriate foundation techniques for low-cost housing in marshy areas. The objective of this research project was broad-based and envisaged improving the current planning, design and construction techniques to achieve significant economy in construction on marshy lands.

The research project, which got off the ground by early 1986, involved detailed field monitoring of 135 existing and new buildings spread over 35 locations in and around the City of Colombo over a period of nearly four years. Continued close co-operation of a number of agencies both in public and private sectors were necessary for successful conclusion of this monitoring work. Despite security problems prevailing in the project area, the detailed field work has now been completed in time, largely due to the commitment and hard work of the project staff. The final report of the research project incorporates the research findings most of which are given in the following chapters of this publication. These include:-

a) A vierendeel girder type of foundation developed and used for the first time in Sri Lanka. This new foundation type, due to its high rigidity, has been proved in field trials to be ideally suited and most economical for constructing low-rise buildings on marshy lands. A number of buildings have already been constructed both in public and in private sectors using this type of foundation and its performance has now been tested for nearly three years.

b) Planning guidelines developed for architects incorporating appropriate structural considerations for low-rise buildings. Use of these guidelines will further reduce the cost of construction through use of rigidity of the superstructure, and thereby eliminate the need for costly foundations. These guidelines have also been successfully used by NBRO in Government projects to substantially reduce cost of foundations.

c) Design guidelines developed for structural engineers to promote rational design of foundations, taking into consideration soil, foundation and superstructure interaction. The guidelines include permissible limiting deformation criteria which have been developed through monitoring the performance of a large number of new and old buildings built on marshy lands. Codes of practice for design of foundations are now being developed by the national bureau of standards using these guidelines.

d) Guidelines prepared for reclamation of marshy lands using controlled compacted fill. These guidelines are already being used extensively by developing agencies both in private and public sectors engaged in reclamation of low-lying areas, for instance, by the Urban Development Authority and the National Housing Development Authority.

e) Standards formulated for evaluation of engineering behaviour of organic soils found in low-lying marshy areas. Typical values of important strength and deformation parameters of organic soils and their co-relation have also been developed. These information will serve as a ready reckoner for practicing engineers.

f) A computerised data base developed exhibiting details of sub-soil profiles and characteristic properties of different types of soils encountered in the low-lying marshy areas in and around the city of Colombo. This data base provides the basis for detailed geo-technical mapping of low-lying areas in future. It can also serve as an invaluable source of information for developers, engineers and consulting architects engaged in building on marshy lands.

The wide sweep and the comprehensive approach of this research project, which has been possible mainly due to keen interest and close support of the Government (several executing agencies of the Government jointly sponsored the local cost of the project), should be a trend setter in the field of geotechnical research in the country. A steering committing, made up of representatives of the leading executing agencies in both private and public sectors, has monitored the activities of the project over the years. Their suggestions, made from time to time, have helped the execution of the project.

The active use of the research findings in the construction industry will considerably reduce the cost of construction, particularly that of low-cost housing, built on marshy lands. It is, therefore, important that all concerned with planning, design and construction of buildings in the low-lying areas are made fully conversant with these research findings. The seminar organised by NBRO provides an excellent opportunity for wider dissemination of these research findings to the construction industry. Participants at this seminar sould make full use of this opportunity through active discussions with the consultants and scientists associated with this research project.

I wish the Seminar a grand success.

PROBLEMS ASSOCIATED WITH DEVELOPMENT OF LOW LYING AREAS

K.S. SENANAYAKE

1.0 INTRODUCTION

1.1 Geographical, Physiological and Geological aspects of Colombo Region

Colombo region situated on the west coast of Sri Lanka falls under the tropical belt with monsoon rainfall conditions experiencing average annual precipitation in the order of 2400 mm. Annual drainage pattern of the catchment is somewhat complex, with main streams having a north-west trend that broadly coincides with the general strike of rocks and other minor streams having near east-west directions. The main waterway through the region is Kelani Ganga, which together with natural streams and a network of canals running through marshy lands, effects drainage of the catchment.

Three principal physiographical regions which are not well defined but that grade imperceptibly are observed as follows;

(a) A north-south oriented coastal belt with minor highlands extending to about 1km to 5 km inland.
(b) Broad flood plains of Kelani Ganga and its tributaries forming a major part of the study area.
(c) Flat bottomed narrow winding valleys surrounded by hills of low relief generally less than 30 m with minor exceptions.

The raised beach along the coast line about 3.5m to 5 m above the present sea level, has provided one of the highly built-up land space along Galle Road. Irregular bodies of water such as Beira Lake, Kotte Lake, Weras Ganga and Bolgoda Lake, and the swamps such as in Kimbulawala, Nawala-Heen Ela and Yakbedda, that exist within a few kilometers inland are considered to be drowned valleys formed due to minor oscillations of the sea level and subsequent closure of their outlets.

The coastal belt consists predominantly of loose unconsolidated sands with bands or layers of clayey and peaty soils. Deposition of the alluvium in the flood plains of Kelani Ganga is not uniform and sands, silts and clays are observed in layers, lenses or interbanded. The hills of low relief interrupted by narrow winding valleys are gently rounded and undulating and are covered with laterite which forms extensive but discontinuous mantel that effectively covers the underlying crystalline rocks.

1.2 Historical Development of Marshes

The native vegetation that covered over the water bodies and swamps have given rise to, on decomposition and accumulation over the centuries, organic deposits whose solid constituents are observed in various stages of decomposition or preservation. The process of decomposition has been retarded as a result of submergence in water and exclusion of air. Pedologically, these cumulose soils are intrazonal hydromorphous soils commonly referred to as peat or muck. In muck, which has a longer geological history, vegetable matter is decomposed to a greater extent, and may be found intermixed with mineral soil constituents as organic clays or organic silts. Well preserved and plainly visible fibres of undecomposed plant remains are found in peat but plant remnants are hardly visible in muck. Owing to its fibrous nature, peat is a spongy light-weight material which absorbs large quantities of water. The type and properties of peats related to the nature of vegetation from which peat is derived and the conditions under which it is accumulated.

The presence of belts of peats and other organic soils observed at considerable depth indicates existence of swamps and marshes in the study area in the Pleistocene and geologically recent times. The largest peat deposit close to the city area is the Muthurajawela marsh extending along the coastal belt north of Colombo. The peat found there has been classified by the Geological Survey Department of Sri Lanka into three groups broadly according to the type of organic matter contained, viz. Shrub and Tree group, Reed and Sedge group and Humus Peat group. With the data available from extensive subsoil investigations carried out, National Building Research Organisation (NBRO) broadly classified peat found in the low lying areas in and around Colombo city into three groups, viz. Coarse Fibrous peat, Fine Fibrous peat and Fine Amorphous peat. Their qualitative physical and engineering characteristics are given in Table.1.

2.0 LOW LYING AREAS IN AND AROUND COLOMBO

2.1 Study Area

Study area discussed in this paper includes the low lying areas located within the Colombo Municipality and some parts of the Urban Council areas of Kolonnawa, Kotte and Peliyagoda. Giving emphasis to the areas which had been considered to be of immediate importance to development activities as identified by the Urban Development Authority and the Sri Lanka Land Reclamation and Development Corporation, a total of about 400 ha of low lying areas broadly divided into 8 zones are included in the NBRO's study. These are identified as Peliyagoda, Orugodawatta, Bloemendahl, Yakbedda, Maligawatte, Nawala, Battaramulla and Kotte zones.

2.2 Drainage and Reclamation

Major part of low lying areas in Colombo region, before reclamation has a ground elevation below the 2 m contour above mean sea level (MSL), with some areas perennially being submerged. The marshy lands through which the poorly maintained drainage canal network runs, offered natural detention basins, giving a great relief at the times of flood distress in the past. Therefore, in the first instance, an important prerequisite in identifying marshy lands for development is to take into careful consideration of this role of the low lying areas. After assessing the probable storm conditions for the future, and leaving approximately 560 ha for flood detention, the planners have identified nearly 530 ha of low lying lands in the study area for development.

3.0 DEVELOPMENT ACTIVITIES

3.1 Present Land Use

Early development of the mini-port city of Colombo, the known history of which dates back to the tenth century, had been limited to the highlands and hillocks in areas such as Fort, Pettah, Hultsdorf and around the Beira Lake. Following the European domination, the importance of this city became more and more significant and its area gradually expanded. Towards the late 1970's commercial, business as well as the administrative activities were more or less limited to the central areas of the city which extended from the sea upto Baseline Road on the east and from Kelani Ganga upto Wellawatte Canal on the south. Practically all the highlands in the vicinity of the city centre were already built-up under land uses such as wholesale and retail trade, warehousing, etc. It is observed that attempts had been taken from time to time in the past to solve the problem of land shortage by reclaiming low lying lands in the prime locations. While certain areas within the city had population densities as high as 900 persons/ ha, the immediate suburbs were found to be only very lightly populated with less than 50 persons/ha in some areas.

TABLE 1. QUALITATIVE PRESENTATION OF PEATS

TYPE OF PEAT / PROPERTY	COARSE FIBROUS	FINE FIBROUS	AMORPHOUS GRANULAR
MOISTURE CONTENT	Medium to Very High	Very High	Medium
PERMEABILITY	Very High	Medium	Low
SHEAR STRENGTH	High ?	Medium	Low
VOID RATIO	Medium to Very High	High	Medium
COMPRESSIBILITY	Medium to High	High	Medium
UNIT WEIGHT	Medium	Very Low	Relatively High

Policy decision of the government in 1977 for the liberlisation of trade, acceleration of Mahaweli development programme and the launching of several other development projects, triggered off many economic and development activities in the city. In catering to these sudden and increased demands, adequate space to accommodate warehouses, light and heavy industries, commercial and business activities, housing and social welfare facilities etc. became an urgent necessity. With only about 10% of the 3733 ha city area were being available as undeveloped land(Table 2) which were mostly low lying, it became inevitable to expand the city, with the shifting of administrative activities to a New Capital City at Sri Jayawardenapura, Kotte, and relocating the congested warehousing facilities etc., that are scattered over the business centres, in new areas to be developed outside the city limits.

TABLE 2. LAND USE BREAK UP - 1977

Land Use	Extent (ha)	% to total area	Extend to be reclaimed by 2001	% to reclaimed area
Residential	1687	45.2	28	7.8
Commercial	201	5.4	119	33.2
Industrial	151	4.0	34	9.5
Public & Semi-Public	465	12.5	65	18.1
Transport	610	16.3	90	25.1
Open space	142	3.8	23	6.3
Developed	3256	87.2	359	100
Undeveloped	379	10.2	20	
Water	98	2.6		

3.2. Proposed Development Activities

The proposed development activities include the shifting of major administrative activities to a new centre in Kotte with a new parliamentary complex and other infrastructure, integrated urban development project at Peliyagoda over a predominantly marshy area of 105 ha, construction of a large number of low cost houses, low rise and high rise office and residential buildings, warehouses and container yards, and setting up of light and heavy industries.

The major types of buildings or structures and their functions and the estimated loads transmitted to the ground are summarised in Table 3. The break down of the extent of land earmarked for planned development by reclaiming the low lying marshy areas is given in Table 4.

Of these, higher priority had been received in Maligawatte zone in Colombo central for housing, Battaramulla zone for administration facilities, Peliyagoda and Orugodawatte zones for industries, and a large extent of marshy lands in these areas are already developed.

4.0 PROBLEMS ASSOCIATED WITH RECLAMATION OF LOW LYING AREAS

4.1 Subsoil Conditions in Low Lying Areas in and around Colombo

The extensive borehole investigations carried out by the NBRO for the Geotechnical Mapping of Low Lying Areas in and around Colombo (Senanayake, 1986) revealed that a significant feature of these subsoils is the presence of organic soils, predominantly peat, distributed over large areas and extending to considerable depths occasionally exceeding 15 m. These organic deposits, being formed by the natural decomposition and accumulation of vegetable matter over the years, possess very high moisture content and high void ratio, and therefore high compressibility and extremely low strength. Some fundamental geotechnical properties of organic soils encountered in Colombo region are discussed by Ameratunga et al (1989).

TABLE 3. TYPES AND LOADS OF STRUCTURES

Type of Structure	Lay out	Type of footing	Column/wall loads
1 RESIDENTIAL			
Single storeyed		Strip footing	25 KN/M
2-storeyed	Load bearing	-do-	65 KN/M
3-storeyed	wall spacing	-do-	115 KN/M
4-storeyed	= 3.0 m	-do-	165 KN/M
2-storeyed framed	3.5 x 3.5	pad footing	360 KN
3-storeyed framed	grid	-do-	580 KN
4-storeyed framed		-do-	820 KN
2 OFFICE			
3-storeyed framed	4.5 x 4.5	-do-	1380 KN
4-storeyed framed	grid	-do-	1900 KN
3 LIGHT INDUSTRIAL		-do-	65 KN
HEAVY INDUSTRIAL		-do-	175 KN

TABLE 4 EXTENT OF LOW LYING AREAS TO BE DEVELOPED

Zone	Extent (ha)	Proposed Reclaimed level (PUFL) (m + MSL)
Peliyagoda	105	1.67
Nawala Heen-Ela	117	2.13
Colombo Central	19	2.44
Gothatuwa	38	2.44
Yakbedda	13	2.44
Madinnagoda	23	2.44
Kolonnawa	19	2.44
Colombo North	89	2.13
Kotte	107	2.13
Attidiya	50	2.13

4.2 Ground Settlement due to Development activities

By virtue of being low lying, these lands are subjected to frequent submergence and, if not perennially submerged, the ground water table exists close to the ground surface. Therefore for any development activity to take place, it is necessary to raise the ground well above the anticipated highest flood level by filling. With the flood levels estimated at around 1.5m to 2.1 m above mean sea level (MSL), and the existing ground in most areas lying at around - 1.0 m to + 0.5 m MSL, earth filling as high as 3 m to 4 m would be necessary to achieve the required elevation. The load imposed by such filling and other subsequent development activities introduce additional stresses in the subsoils which are already undergoing settlement under their own weight. Due to the highly compressible nature of organic soils, particularly of peat, these settlements could be of a very high order and moreover could continue over a prolonged period. Further, due to variations in vertical profile and lateral distribution of subsoils or due to variations in the loads imposed by development activities, ground behaviour becomes complex resulting in differential settlement in structures supported on reclaimed ground. A major problem encountered in developing low lying areas is the excessive differential settlements that lead to distress or even failure in structures supported on poor soil conditions.

Perhaps in cognition of these problems and construction difficulties anticipated, apparently, the building activities on marshy lands had been deliberately avoided to a great extent in the past. But simple ground improvement methods or special, though costly, foundations for example, rafts or piles had been adopted where construction of structures was unavoidable. However, with the current growing pressure for land, a large extent of low lying areas had been developed hurriedly and hapzardly in the recent past, and many of these structures constructed over the reclaimed land have undergone mild to severe distress. This was mainly due to the lack of information and knowledge on the behaviour of complex organic subsoils, particularly the peaty soils prevalent in the marshy areas, in responding to the development activities.

Damages due to ignorance is costly and often irreparable. Therefore, special attention is directed in the present study undertaken by NBRO on attempts to understand the short term and long term behaviour of the developed land and the performance of structures built on it through field and laboratory investigations and, theoretical and empirical observations.

4.3 Problems of Haphazard Development

Whereas large areas of state owned/state vested low lying lands are being considered for planned development, apparently there are no legislations effectively implemented on proper development of marshy lands owned by individuals. Often due to inadequacy of guidance by the state and other relevant bodies or due to financial constraints, private house builders and developers seldom seek professional advice on land development or construction of buildings in marshy areas. Attempts for saving by avoiding expert advice or essential subsoil investigations may usually end up in costly repairs to the structures.

One aspect the un-advised builder will be ignorant of is the fact that reclaimed ground can undergo prolonged settlements, which if high not only cause distress in the buildings, but also could bring the developed ground surface eventually to an elevation much lower than the recommended ultimate ground level which is essentially dependent on the highest flood level anticipated in the area concerned. Another aspect is the necessity to provide proper drainage from the developed land. Natural drainage is sometimes blocked by fillings made for temporary access roads etc., Negligence to provide proper storm drainage, blocking or narrowing down of natural drainage channels by earth filling etc., combinedly can cause a building to undergo submergence or the sewage facilities to malfunction. Staggered development in a particular area, may aggravate this situation, specially when the late-comers to build try to alleviate such problems by raising the ground higher than in the adjoining lands.

The high cost of land delimits the extent of building lots that could be acquired within economic reach of most private builders. The performance of structures built on small narrow lots of land can be severely affected by the loads imposed by subsequent development activities taking place in adjacent lands. Sometimes, the building walls constructed close to or along the land boundary or the boundary walls can be undermined by excavations in a neighbouring lot. Again if filling is done at two different levels, in adjacent lands, a wall along the common boundary, which has not been designed as a retaining wall will be subjected to danger. One could imagine the plight of a building constructed very close to the edge of a high filling adjacent to a deep drain or canal or an undeveloped marsh.

Distress of tilting, distortion and cracks etc., and even functional failures or threat to structural safety in such buildings due to haphazard and staggered development can be avoided to a great extent by proper guidance. Collective simultaneous development in such neighbouring small plots should be encouraged and if this is possible, precious resources could be effectively utilized. Therefore, time has now come for the building and construction industry to intervene and advice the builders so that a colossal loss in national economy can be avoided. In this respect, it would be necessary for the state authorities responsible for land use planning and development to ensure that reclamation and development of all marshy areas is carried out scientifically and in a controlled manner, for example by acquisition and vesting of all marshy small holdings with the state until they are released to prospective builders or back to the original owners as properly developed lands along with information on subsoils etc., and guidelines on their optimum use. Such guidelines could include possible and most appropriate building types, scale of structures, recommended foundation types etc.

4.4 Problems Associated with Planned Development

Large extents of low lying areas had been earmarked for planned development , for example in Peliyagoda and Orugodawatte in the northern part of the Study Area. Mainly due to logistical and economic reasons, development in such vast areas has to take place in stages and over several years, if not in a decade or so.
The planner, from his point of view, prefers to provide an ideal layout for the selected land use. In this process, it is quite possible that he may overlook or disregard the various development thresholds associated with the project. Importance of drainage and geotechnical aspects may not be critical in developing highlands with sound subsoil conditions. However, in utilizing low lying areas in Colombo region for economic activities one must seriously consider both drainage and ground conditions giving highest importance. Of course, the necessity for assessing the environmental impact need to be highlighted. If the geotechnical and environmental problems are not sufficiently visualised, eventually the project may not bring in the envisaged results.

As the first step of developing a low lying area a suitable ground formation should be established. In order to prevent the developed land from inundation, its ultimate (long term) formation level should be determined based on the possible highest flood level derived from hydrological analysis. However, if allowance is not made for the possible long term settlement of the underlying soil strata due to loads imposed by development activities in the determination of the thickness of fill required, the developed land is likely to end up at an elevation sometimes even below the actual flood level as settlement continues with time. The planned ultimate formation level (PUFL), which is a long term requirement, is likely to be easily mistaken for the final filled/reclamation level, which is the ground level on completion of filling, or reclaiming. This could lead to extensive irreparable damages specially where large scale development activities are undertaken with colossal investment if ground subsides below the ground water table or the flood level severely effecting the functions of the facilities.

Therefore it is important to accommodate the geotechnical characteristics and natural features of the area to be developed in land use planning, for example, by identifying the least, moderately and most problematic areas, so that land use can be optimised with proper selection of the facilities suitable for each area. The deepest parts of the marsh, i.e. where weak soils extend the deepest, may be more suitable for lakes or detention basins, unless, heavy structures, that are anyway intended to be supported on deep foundations, are to be located. Similarly the peripheries of marshy lands where strong ground is available at shallow depths may be utilized suitably with lesser foundation costs. To carry out such planning preliminary geotechnical investigations are essential. Such geotechnical information gathered would be also useful to the developers at later stages. Much valuable supplementary information could be gathered from old may of the area and by inquiring the inhabitants of the area.

A reclaimed land could be effectively utilized if it is properly planned. Haste could be waste particularly with respect to developing low lying areas. A land after reclamation, if could be patiently utilized for an interim purpose, for example, as a storage yard, container yard, or a park for heavy machinery etc., until the ground stabilizes to a suitable level, it would be possible to accommodate the permanent development activities at lesser cost. Roads through marshy lands constructed in a hurry, can undergo significant settlement and therefore would require repetitive repairs and maintenance . It would therefore be preferable to invest on costly pavements and finishing works of roads only after the embankments are stabilized.

In the research project undertaken by the NBRO with the objective of "evolving appropriate foundations for construction, particularly for low cost housing in reclaimed marshy areas", it was revealed that over a decade ago, the number of permanent buildings that had been constructed in the low lying marshy area was small, and these too were located in the peripheral lands. However, a large number of buildings supported on different types of conventional and special shallow and deep foundations, now exist on reclaimed lands.

It is encouraging to know that many innovative foundation designs had been tried by the engineers in Sri Lanka in the recent past, to overcome problems in the low lying areas. Some of these foundations are observed to have so far performed satisfactorily even though their mechanism of functioning appears to be very much complex.

A major advancement in the appropriate foundation techniques for reclaimed marshy lands be hopefully anticipated through development of innovative methods, adoption of techniques tried and tested in other countries under similar situations (e.g. Cakar Ayam foundation developed in Indonesia) and by continued monitoring of performance with a view to improve the techniques.

A technique for installation of under-reamed piles through organic soils is being developed by the NBRO and the results so far observed indicate that this foundation type will have a great potential in Sri Lanka.

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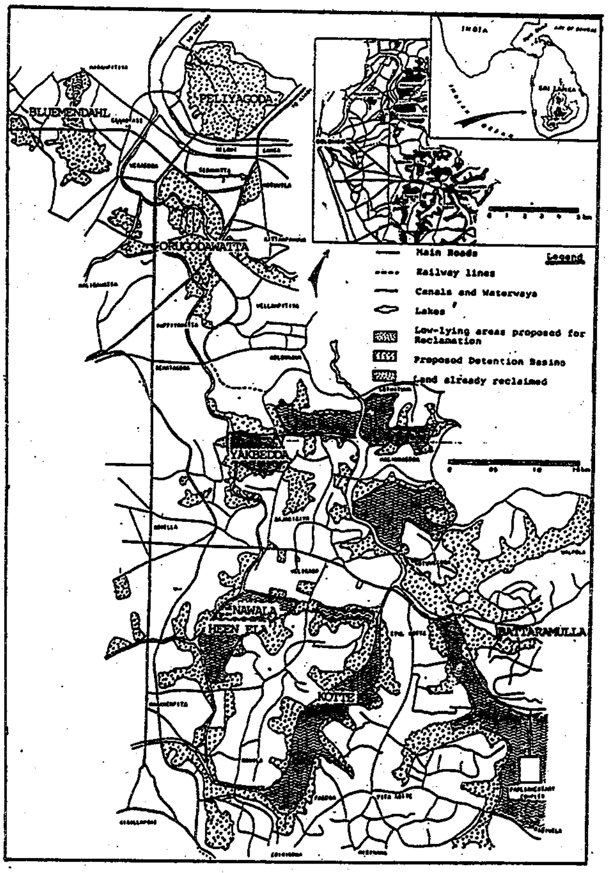
SUBSOIL CHARACTERISTICS IN LOW LYING AREAS

J. AMERATUNGA, K.T.R. LAKSHMAN, S. GANESAMOORTHI AND N. KUGANENTHIRA

1.0 INTRODUCTION

Distribution of low lying areas in and around Colombo are given in Fig. 1. The total area of low lying lands in and around Colombo is more than 1000 hectares. Under the accelerated development work that were embarked in the late seventies it was planned to develop all low lying areas by the end of this century. Under this programme, immediate attention was focused on approximately 800 ha of low lying marshy areas within Municipal Council limits of Colombo,Kotte, Kolonnawa and Peliyagoda (Fig.1). For convenience they have been divided into 8 zones as proposed by Senanayake (1986). These zones are as follows (see Fig. 2).



DISTRIBUTION OF LOW-LYING AREAS IN AND AROUND COLOMBO

FIG. 1

1. Peliyagoda Zone
2. Orugodawatta Zone
3. Bloemendhal Zone
4. Yakbedda Zone
5. Battaramulla Zone
6. Nawala Heen Ela Zone
7. Maligawatta Zone
8. Kotte Zone

These low lying marshes consist predominantly of organic subsoils. These soils have been formed due to decomposition and accumulation of native vegetation that had covered water bodies, over a period of time.

The ground surface in low lying areas is generally below the contour of + 2m above mean sea level. Ground water table in these areas is either at or very near the ground surface or in some areas above the ground surface.

2.0 INVESTIGATIONS CARRIED OUT

Investigations had been carried out in low lying areas since early eighties by the Central Soils Laboratory (CSL) of the Urban Development Authority (UDA), which in 1984 became the National Building Research Organisation (NBRO) with the amalgamation of the Building Research Establishment (BRE) of the State Engineering Corporation (SEC). While some of these investigations have been carried out as routine consultancy work, major part of the investigations used in this study have been carried out for specific projects viz.

a) Soil Investigations for the Integrated Urban Development- Peliyagoda (IUDP) Stage I
b) Soil Investigations for the Research Project on "Geotechnical Mapping"
c) Soil Investigations for the Research Project on "Appropriate Foundation types for Low Lying Areas" (LLA)
d) Soil Investigations for the Research Project on "Under Reamed Piles" (URP)
e) Soil Investigations carried out for consultancy projects.

The total area covered in each zone is given in Table 1.

Zone	Area covered ha	I.U.D.P. stage I	Geotechnical Mapping	L.L.A. project	U.R.P. project	Consultancy projects
Peliyagoda Zone	78.92	*	*	*	*	*
Orugodawatte Zone	43.43		*	*		*
Blomendhal Zone	42.0		*	*		*
Yakbedda Zone	37.81		*	*		*
Battaramulla Zone	25.9		*	*		*
Nawala Heen Ela Zone	165.98		*	*		*
Maligawatte Zone	81.0			*		*
Kotte Zone	156.6			*	*	*

TABLE 1

ZONEWISE DISTRIBUTION OF INVESTIGATED AREA UNDER EACH PROJECT

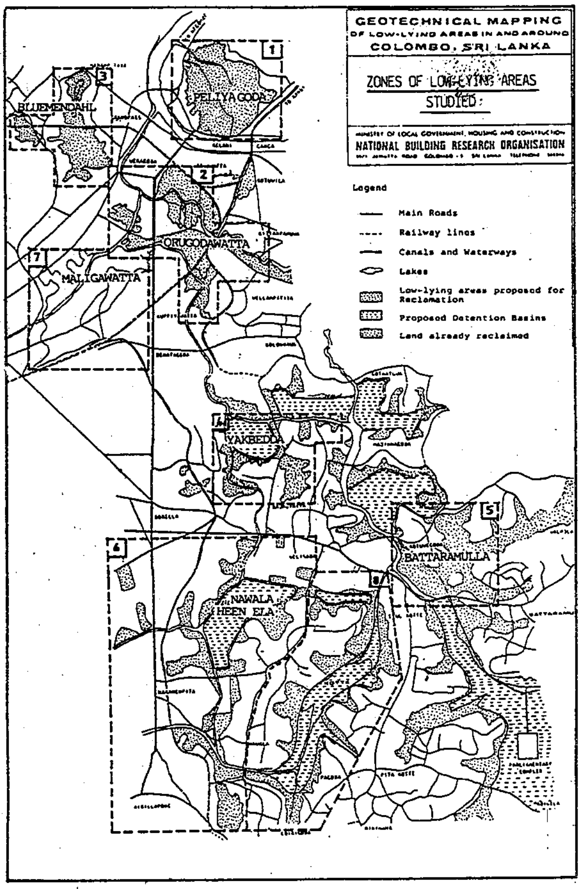


FIG. 2

2.1 Field Investigations

Field investigations carried out in these areas include :

1) Advancing of Boreholes and Auger holes
2) Sampling
3) Carrying out Standard Penetration Tests (SPT)
4) Carrying out Vane Shear Tests (VST)
5) Carrying out Static Cone Penetration Tests (SCPT)
6) Carrying out Field Density Tests

2.1.1 Advancing of boreholes and auger holes

Majority of the boreholes had been advanced using wash boring technique. Bentonite slurry had been used whenever necessary to keep the borehole stable. In some cases casings have to be used because bentonite proves ineffective when very loose sandy soils are encountered. Deep boreholes have generally been terminated at a weathered rock stratum or a hard stratum.

Auger holes have been advanced using standard 3", 4" and 6" augers having different auger heads, each for specific type of soil. Augers are generally used to confirm the topmost strata only. The maximum depth investigated using augers had been 10m.

2.1.2 Sampling and storage

Undisturbed soil samples were obtained using thin walled samplers of 38mm, 50mm and 70mm diameter, while disturbed samples are collected from the split spoon sampler of the Standard Penetration Test (SPT) or while augering. It is very important to ensure that the area ratio of the sampler is very low. At shallow depths, especially in peat, box sampling also had been carried out.

It is important to maintain moisture equilibrium of undisturbed samples. Samples should be stored in humidity-controlled rooms. Block samples should be periodically inverted and, horizontally stacked tube samples should be occasionally turned. Even during extrusion, it is preferable to hold the sample tube in the horizontal plane.

Collection of ground water is carried out within boreholes with the use of a bailer. Water is collected in a steralised bottle and immediately sealed at the site.

2.1.3 Field tests

Over recent years, there have been considerable development of in-situ testing because of dissatisfaction with more traditional methods. In-situ tests become appealing especially due to difficulties and disturbances in sampling. Further, the performance of a structure is governed by the whole mass characteristics of the soil and therefore during in-situ testing, a more representative sample will be tested. However it must be stated that in most field tests boundary conditions cannot be properly controlled and there is no theoretical rationale.

The field investigations carried out for this study included.

(1) Strength tests
   a) Standard Penetration Test
   b) Static Cone Penetration Test
   c) Vane Shear Test

(2) Field Density Tests

2.1.3.1 Standard Penetration Test (SPT)

Standard Penetration Tests were carried out within boreholes at regular depth intervals (generally 1 m) to obtain a continuous strength profile. SPT-N value, the number of blows necessary to push the SPT split spoon sampler the last 30cm portion of a 45cm length advance, is recorded during the test. Disturbed samples collected within the SPT tube are used for laboratory testing purposes. SPT is essentially developed for cohesionless soils but it had been extended for cohesive soils, although the reliability is highly doubtful. The interpretation of SPT is purely empirical with N-value related to friction angle, relative density and Young's modulus.

2.1.3.2 Static Cone Penetration Test (SCPT)

In this test, a cone with a 60^{o} apex angle and a cross section of 10 cm^2 is pushed into the ground mechanically and measurements of end resistance (q_c) and the side friction (q_s) are obtained. Correlations have been made with q_c and, bearing capacity, settlement, N-value, undrained shear strength and deformation modulus. However, these correlations appear to be much dependent on soil type. For Sri Lankan soils such correlations had not been established in the past, possibly because of lack of information gathered from this method of testing.

2.1.3.3 Vane Shear Test (VST)

Vane Shear Test is preferred over other in-situ tests to measure the undrained shear strength of soft clays. It has been used in fibrous peat too in other countries, although the success is doubtful.

In the VST, the vane is driven to the desired depth and rotated at a constant rate and the torque is measured. Unlike the SPT, VST has a theoretical rationale, and therefore, the undrained shear strength can be interpreted from the measured torque.

2.1.3.4 Field Density Test

Field density test is important, especially in reclaimed lands, to obtain the density of the fill and the percentage compaction achieved. Field density tests are generally carried out by NBRO using two methods viz.

1) Sand Cone Method
2) Core Cutter Method

In the sand cone method, approximately 12 cm diameter hole is made in the fill to a depth 20 cm, and the soil excavated is weighed and the moisture content determined. By obtaining the weight of 'Standard' sand of known density necessary to fill the hole in the ground, the volume of the hole and therefore density, of the soil can be calculated.

In the core cutter method a cylinder (diameter = 10cm) is driven to a depth of around 14cm. The sample collected is weighed and the volume and moisture content determined. This is useful when testing soil at deeper depths.

2.2 Laboratory Investigations.

Laboratory investigations are carried out on disturbed and undisturbed samples collected in the field. Although there are disadvantages associated with laboratory testing, as mentioned in Section 2.1, one advantage is the possibility of controlling the boundary conditions with a high degree of precision. Further, laboratory approach permits an examination of variety of other conditions which may be relevant to future changes under structural loads. Anyway as far as economy is concerned, laboratory tests may be comparatively cheaper.

Laboratory investigations that had been carried out for this study include :

1) Natural Moisture Content Tests
2) Atterberg Limit Tests
3) Grain Size Distribution Tests
4) Consolidation Tests
5) Strength Tests
6) Compaction Tests
7) Organic Content Tests

2.2.1 Natural Moisture Content (w)

To determine the moisture content in organic soils, standard method is to oven dry the sample around 110^{o} C for 24 hours. The loss of weight during drying, as a percentage of the remaining weight, is defined as the moisture content. However, in organic soils, it is feared that some amount of organic matter may undergo loss of weight due to oxidisation at 110^{o} C. There are various suggestions and recommendations on the appropriate temperature for oven drying of organic soils. However, only little research had been carried out in Sri Lanka.

At NBRO, the effect of oven temperature and period of drying for organic soils was studied. Samples were tested at three drying temperatures of 110^{o} C, 80^{o}C and 65^{o}C (see Fig.3).In Fig.3(a), temperature is maintained at 65^{o} C for three days and raised to 110^{o} C. In Fig.3(b), temperature is maintained at 80^{o} C for 5 days and raised to 110^{o} C. In Fig.3(c), temperature was maintained at 110^{o} C. Readings were taken every 24 hours to determine the moisture content. It is evident from the figures, that only at 65^{o} C, the weight remains constant with time. Therefore, 65^{o} C appears to be most appropriate for organic soils.

2.2.2 Atterberg Limits

Atterberg limits, simple indices for fine grained soils are considered very important in the classification of soils and during preliminary designs. These limits are correlated with important parameters such as shear strength, compressibility etc. and proves very useful for the designer. These limits cannot be performed on fibrous peat samples.

The liquid limit test is usually carried out by Casagrande's percussion cup method or Swedish fall cone method. Their relative merits/demerits are given by Karlsson (1981), and Sherwood and Ryley (1970). Instead of carrying out several tests to determine the liquid limit, Waterways

Experiment Station (1949) suggested the one point liquid limit test which requires only a single determination of the moisture content to obtain the liquid limit. The liquid limit (w_L) is found by expressions relating w_L to moisture content and the number of blows (N). Many of these relations are of the following forms.

$$w_L = w\ (N/25)^n$$
$$w_L = w\ [1/(a-b \log N)]$$

where a,b and h are constants for the soil.

From 42 samples collected by NBRO from Battaramulla, Nawala, Orugodawatte, Peliyagoda and Bloemendhal zones a study was conducted to obtain empirical relationships for organic soils in Colombo and suburbs for the one point method (Lakshman & Sivakugan, 1987). The following correlations were then derived with excellent agreement.

$$w_L = w\ (N/25)^{0.14}$$
$$w_L = w/[1/(1.43 - 0.31 \log N)]$$

2.2.3 Grain Size Distribution

In granular soils, the grain size distribution has significant influence on the engineering properties. In cohesive soils too, the percentage of clay/silt/sand particles could drastically change the properties exhibited by the sample. Considering the nature of peaty soils, the Authors consider it most appropriate to limit the grain size analysis for inorganic particles.

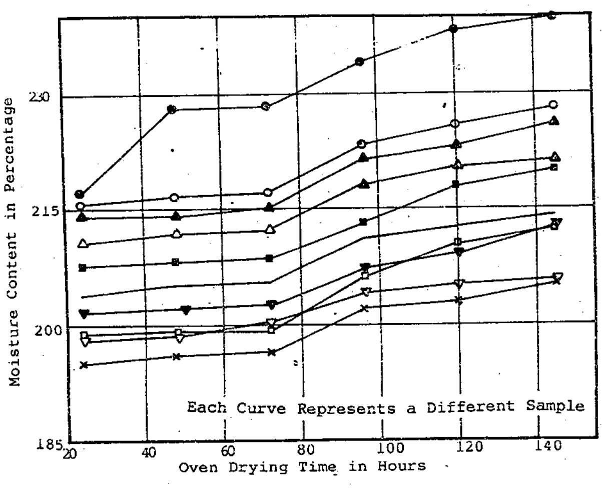


FIG 3 (a) - 65° c for three days and later 110° c

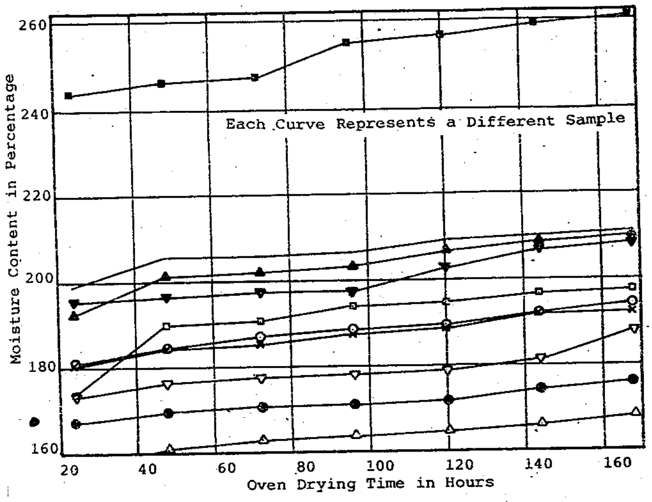


FIG 3 (b) - 80° c for five days and later 110° c

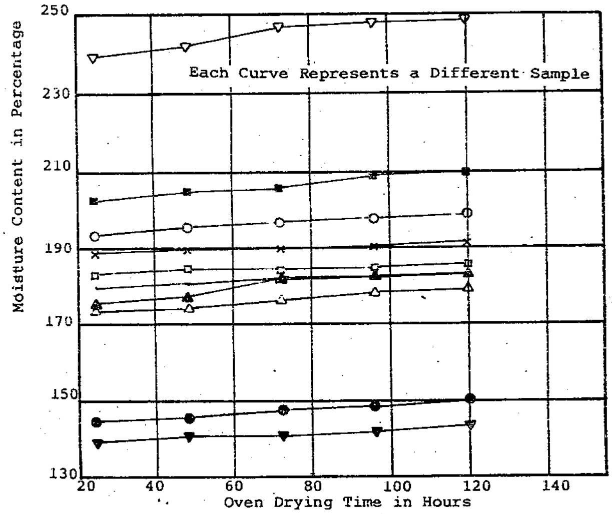


FIG 3 (c) - at 110° c

2.2.4 Consolidation Test

Consolidation tests are exclusively performed on cohesive soils to determine the compressibility characteristics of cohesive soils. The results of this test are essential to carry out settlement analyses in cohesive soils. The important parameters derived from this test and their use are listed below.

(i) Prediction of Primary Consolidation Settlement
- Compression Index, C_c
- Recompression Index, C_r
- Preconsolidation Pressure
- Coefficient of Volume Compressibility (m_v)

(ii) Prediction of Rate of Settlement
- Coefficient of Consolidation (c_v)

(iii) Prediction of Secondary Compression
- Coefficient of Secondary Compression (c_α)

Standard Consolidation test is generally carried out on 5cm-7cm diameter cylindrical undisturbed samples of about 2cm high, in an oedometer. The effect of friction of the sides of the specimen can be minimised by applying a lubricant or by using a sample of larger D/H (Diameter/Height) ratio. The latter is most desirable when testing peaty soils.

The interpretation of the test is based on Terzaghi's 1-D consolidation theory. It has been found that 1-D consolidation curve could be deeply affected by disturbances (Shown in Fig.4). Further, in very soft soils, the Load Increment Ratio (LIR) plays and important role on the shape of the consolidation curve (see Sivakugan,1986)(LIR is defined as the ratio of increment of load to current load). For consolidation tests on organic soils, at NBRO LIR of 1 is used. Also, the smallest load applied is kept around 0.05 kg/cm^2 to obtain a more accurate prediction.

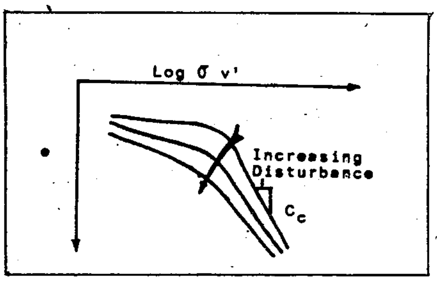


FIG 4 - RELATIONSHIP OF LOG PRESSURE Vs VOID RATIO

2.2.5 Strength Tests

Triaxial test is generally regarded as the most appropriate laboratory test to determine strength parameters. As the complete stress strain curve can be obtained in this test, in addition to strength parameters, deformation parameters can also be obtained. Triaxial tests are carried out on both undisturbed and remoulded samples. For cohesive soils, the following tests can be carried out:

(i) Unconsolidated Undrained (UU) Test or Quick Test
(ii) Consolidated Undrained (CU) Test
(iii) Consolidated Drained (CD) Test

Although both stress controlled and strain controlled type tests are possible, for strain softening soils, peak behaviour can be studied only in strain controlled tests. For testing of organic clays and peats described in this paper only strain controlled tests were used.

Unconfined compression test can be considered as a special case of the triaxial test with no lateral confinement (i.e. lateral pressure is zero). As the test is very simple and quick, and also because the complete stress strain curve can be obtained, it is widely used instead of the triaxial test. However, due to disturbance effects, the same reliability as from triaxial tests cannot be expected. From a limited number of tests carried out at NBRO it is seen that the modulus obtained from UU test could be more than three times than that obtained from a UC Test (Sritharan et al, 1989). This confirms the work of Crawford & Burn in 1962 as reported by Bowles (1982).

Laboratory vane shear test is another method of obtaining the shear strength of cohesive soils. This becomes useful when extrusion is difficult and other types of laboratory shear strength tests cannot be carried out. Table 2 shows a comparison of shear strength using different laboratory tests carried out at NBRO.

Soil type	Moisture Content	Cohesion - S_u (Kg/cm^2)		
		Lab Vane Shear Test	Field Vane Shear Test	Unconfined Compression Test
Pt	693	0.117	0.110	0.024
	486	0.110	0.170	0.015
	449	0.128	0.110	0.009
	291	0.291	0.460	0.030
	220	0.297	0.260	0.070
	197	0.134	0.290	0.085

Table 2

SHEAR STRENGTH VALUES OBTAINED FROM DIFFERENT TESTS

2.2.6 Compaction test

The compaction test is performed to assess the adequacy of compaction of a fill. It is checked by obtaining the field dry density (by Field Density Test) and comparing with the maximum density (Standard Proctor density) that can be achieved by compacting it in the laboratory using a standard procedure. The standard procedure involves the compaction of soil, layer by layer (in 5 layers), in a standard mould, by giving 27 blows per layer with a 2.5kg hammer drop of 30cm.

2.2.7 Ash Content/Organic Content

Although peat consists predominantly of vegetable matter, it does contain various inorganic materials, sometimes referred to as "ash" on ignition. Ash content is defined as the inorganic residue which remains after the ignition of the combustible material within a sample by prescribed quantitative methods.

$$\text{Ash Content (\%)} = \frac{\text{weight of ash/residue} \times 100}{\text{dry weight of original sample}}$$

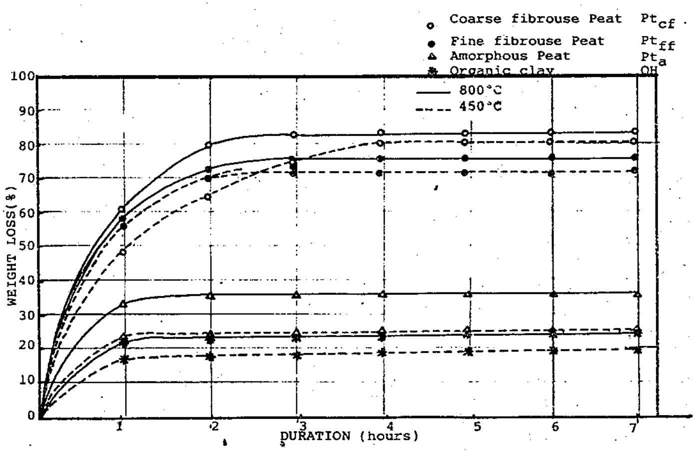
Organic Content (%) = 100 - Ash Content (%)

In the firing process, matter other than the organic carbon may be burned, and thus the above approximation can be in error by 5% to 15%.

It is well known that the organic content greatly influences the engineering behaviour of subsoils. Therefore it becomes important to obtain the correct organic content for a soil.

Although there are several methods, the most widely used method, and adopted at NBRO, to determine the ash/organic contain is to fire an over dried sample in a muffle furnace until the soil has been reduced to an ash. The temperature and the length of firing vary. Although Muskeg Engineering Handbook recommends a temperature of 800°C to 900°C, according to Landva et al (1983), Arman has recommended low temperatures claiming loss of mineral matter. However according to Landva et al, Silfverberg who had carried out a comprehensive investigation on the influence of organic matter in Differential Thermal Analysis (DTA) of clays, and of the chemical determination of soil organic matter, had concluded that a temperature of at least 850°C is required for removal of the less oxidisable parts of the peat, such as cellulose, proteins, fats etc. Silfverberg also recognised that the high temperatures can, besides oxidising the organics, also cause mineral decomposition.

A study was carried out at NBRO to determine the correct temperature and duration for the ignition test (Sritharan et al 1989). Samples of coarse fibrous peat, fine fibrous peat, amorphous peat and organic clay were fired at two different temperatures i.e. 800°C and 450°C. Results are shown in Fig.5. Minimum time required to attain a constant weight for different temperatures are shown in Table 3.



RESULTS OF IGNITION TESTS IN ORGANIC SOILS AT DIFFERENT TEMPERATURES

FIG 5

Soil type	Minimum duration for complete oxidization (Hrs) Temparature	
	800° c	450° c
Organic Clay	1.0	1.0
Amorphous Peat	1.0	1.0
Fibrous Peat	2.5	4.0

Table 3

MINIMUM DURATION FOR COMPLETE OXIDIZATION

3.0 FORMULATION OF SOIL SERIES

In 1984, NBRO launched a research programme to carry out Geotechnical Mapping of low lying areas in Colombo and suburbs. The area considered for this study included Colombo City and some parts of the Urban Councils of Peliyagoda, Kolonnawa and Kotte. From results obtained from a total of 228 boreholes and auger holes, Senanayake (1986) proposed a soil series called M-Series, named after Muthurajawela marsh (an area situated on the north of Colombo and has the largest peat deposit in the region). This series superseded the earlier one proposed by Ray and others (1982) in a project report on "Geotechnical Engineering Problems Associated with Development of Low-Lying Areas", prepared by the Central Soils Testing Laboratory of the Urban Development Authority. The M-Series having five subseries has been formulated according to the existence or non existence of peat horizons (For the purpose of identifying the soil series organic soils i.e. organic clay, organic silt and peat, are all included under peat) and the number and order of sequence of the peat horizons.

The subseries M-I to M-V identified is shown in Table 4. As indicated in the table, M-V series consists of a single inorganic horizon up to the hard stratum. This profile is located close to the periphery of a marsh or when hillocks/highlands protrude into the low-lying lands. M-I series is formed when a single peat horizon overlies an inorganic horizon extending to the hard stratum. M-II and M-IV Series have two peat horizons that are considered to be the result of sea oscillations. A single peat horizon is sandwiched between two inorganic horizons in the MIII series. Range of depth and thickness of each horizon in MI-MV series is given in Table 5.

4.0 CLASSIFICATION OF PEAT AND OTHER ORGANIC SOILS

There are many classification systems used by engineers to describe peat. However, on reviewing the literature, it becomes evident that most of the existing classification systems are based on the use of peats and organic soils as a growing medium in horticulture, agriculture and forestry, or as a fuel. Hence some botanical knowledge is involved. The Geological Survey Department of Sri Lanka has attempted a similar classification with greater emphasis on parent vegetation as reported by Ray et al, 1986. This classification is as follows.

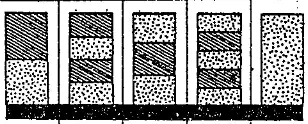
1. Shrubs and tree groups
2. Reed and sedge group
3. Humus peat

McFarlane (1969) had described three different types of peat based on their predominant structural characteristics.

1. Coarse fibrous peat. This consists of a woody coarse fibrous open structure containing woody and non woody particles.

2. Fine fibrous peat. This is an intermediate type where the mass is predominantly fine fibrous, containing woody and non woody particles.

3. Amorphous granular peat. The state of decomposition is very advanced and the soil particles are mainly colloidal with traces of fine fibres.

Subseries			M-I	M-II	M-III	M-IV	M-V
Number of horizons			2	4	3	5	1
Number of peat layers			1	2	1	2	0
Schematic representation Peat Horizon Inorganic Horizon Hard Stratum							
Distribution of Boreholes	Zone	Total					
	Peliyagoda	70	7	3	46	10	4
	Orugodawatta	40	1	0	19	20	0
	Bloemendhal	4	2	0	2	0	0
	Yakbedda	17	2	0	5	1	9
	Nawala	39	2	1	15	6	15
	Battaramulla	9	7	0	2	0	0
	Kotte*	22	-	-	-	-	-
	Maligawatta	27	2	0	23	0	2

* Shallow boreholes (Augerholes) are not classified.

TABLE 4

MUTHURAJAWELA SOIL SUB SERIES

There are several classification methods for peat and other organic soils based on the organic content and ash content. Fig.6 shows divergent opinions on the definition of peat using ash/organic content. Lakshman (1988) proposed that soil should be identified as organic if the organic contain is more than 20%. The reason for this suggestion is because void ratio Vs log (pressure) curve in the consolidation test becomes nonlinear in samples containing more than 20% of organic content.

The classification of organic soil on the basis of the organic content alone would not be sufficient for the geotechnical engineer, partly because of the doubts associated with the "standard test procedures" and partly because it would not include any information on the nature of the inorganic materials contained in the soil.

Subseries	Horizon	Soil Type	Depth of Horizon(m)	Thickness of Horizon (m)
M-I	1	Peat	0-10.5	1.0-10.6
	2	Inorganic	2.0-21.0	2.5-17.5
M-II	1	Peat	0-6.0	2.7-6.0
	2	Inorganic	2.2-9.4	1.7-3.4
	3	Peat	4.5-11.0	1.3-6.8
	4	Inorganic	11.0 and below	not fully penetrated
M-III	1	Inorganic	0.5-12.2	0.5-12.2
	2	Peat	0.5-20.0	0.5-16.0
	3	Inorganic	3.5-26.0	5.0-22.5
M-IV	1	Inorganic	0-6.5	1.0-6.5
	2	Peat	0.7-14.0	0.7-13.0
	3	Inorganic	1.7-18.0	1.0-15.2
	4	Peat	4.0-20.0	0.5-7.0
	5	Inorganic	5.0-22.5	1.0-9.0
M-V	1	Inorganic	0-14.5	0-14.5

TABLE 5

RANGE OF DEPTH AND THICKNESS OF EACH HORIZON IN MI_ MV SUB SERIES

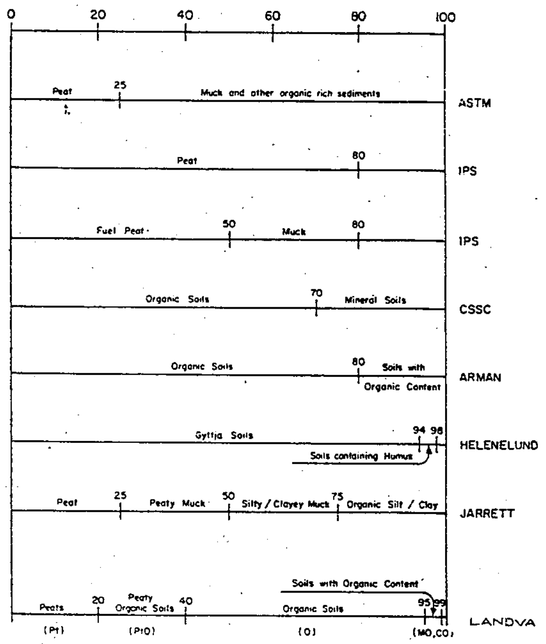


FIG 6 - DEFINITION OF PEAT USING ASH CONTENT after LANDVA and ROCHELLE (1983)

According to the Unified Soil Classification (USC) system, all soils are grouped into coarse-grained, fine-grained, and highly organic (peaty) soils. Fine-grained group is further sub-divided to inorganic clays, inorganic silts and, organic silts and clays. These sub groups are identified on the basis of index tests and Atterberg Limits. It is significant to note that all organic clays and silts are located below the A-line on the plasticity chart. In fact, according to the guidelines given by Terzaghi & Peck (1967) to identify organic soils, it is suggested that, if the liquid limit decreases by 30% or more due to oven drying from that of a fresh air dried sample, to classify the soil as organic. However, in marginal cases this may prove to be difficult. In such instances the organic content would be more useful.

Another instance where organic content is useful is when the sample is amorphous granular. In many instances it is difficult to differentiate between amorphous granular peat and organic clays/silts by visual classification. In such instances, the organic content/ash content proves to be useful. Fig. 7 shows the relationship between ash content and water content as reported in the Muskeg Engineering Handbook (1969).

5.0 ENGINEERING BEHAVIOUR OF ORGANIC/PEATY SOILS AND USEFUL ENGINEERING DATA

A typical area consisting of peaty/organic sub-soils could have different properties both in the vertical and in the horizontal direction. Variations could be due to the type of peat/organic soil and its formation, to degree of decomposition of vegetation, and to moisture content. Large variations could be found even within very small areas. Results obtained from tests conducted at a certain location therefore, may not likely to be representative of other locations and depth. For example at a site at Peliyagoda, a warehouse of 140m length constructed over an area, assuming a generally homogeneous soil profile, was later found to have undergone severe distress because an underlain peat layer varied from 3m to 10m in thickness along the length of the warehouse. In such a case, frequent and inexpensive sampling and classification would have yielded the range of variations within the area under investigation.

Despite the non-homogeneous nature of peat deposits, it is still useful to have a theoretical concept of the behaviour of peat, just as it is for inorganic soils. For example in peaty soils, unlike in inorganic soils, change in permeability during the consolidation process and the high order of rate of creep are significant. Although several theories have been developed on different assumptions to explain the behaviour of peat, unfortunately they are not strictly valid for all peats. For example considering the complexity of consolidation of peat, the disturbances and inaccuracies resulting during sampling and testing, non-homogeneous nature of peaty terrain, it may not be appropriate to apply rigorous analytical theories to obtain settlements for routine calculations. The "old" theory of Terzaghi, which is very much simpler, could be equally valid in such situations.

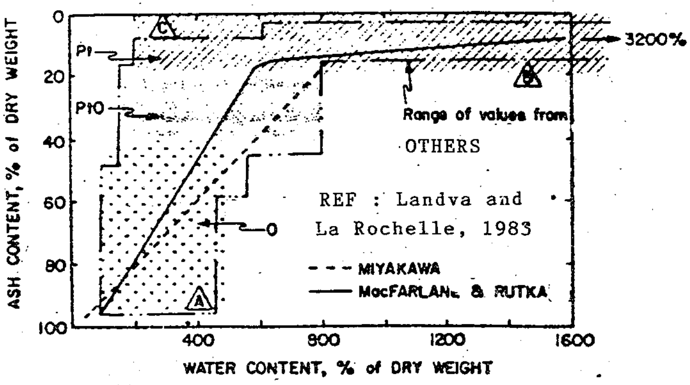


FIG 7 - RELATIONSHIP OF ASH CONTENT Vs MOISTURE CONTENT

In fact, in most instances, appropriate values and simple theories or simple tests are more than adequate for obtaining necessary parameters especially in preliminary design stages. In the following sections typical values and correlations for peats and organic soils will be discussed. Further, Table 6 summarises typical ranges for important parameters.

Natural moisture content (w) is one of the most useful characteristic of organic soils which could be easily determined. High w values are generally associated with peaty soils. The variation of w in peaty soils could range from as low as 50 to as high as 550%.

Atterberg limits are also important indices for organic soils. As seen from Table 6, the plastic limit could vary from 30 to 70 while the liquid limit ranges from 60 to 200.

PARAMETER	VALUE
Moisture Content (w) %	50 - 550
Liquid Limit (w_l) %	60 - 200
Plastic Limit (p_l) %	30 - 70
Specific Gravity (G_s)	1.7 - 2.3
Insitu Void Ratio (e_0)	1.0 - 4.0
Insitu Dry Density (γ_d) g/cm³	0.2 - 1.0
Compression Index (C_c)	0.2 - 4.0
Coefficient of Consolidation (C_v) cm²/sec	10^{-3} - 10^{-4}
Coefficient of Volume Compressibility (m_v) cm²/kg	0.05 - 0.7
Coefficient of Secondary Compression C_α	0.002 - 0.05
(E_u/C_u) Unconfined Compression Test	25 - 80

Table 6

TYPICAL VALUES FOR GEOTECHNICAL CHARACTERISTICS OF ORGANIC SOILS IN LOW LOW LYING AREAS

The bulk density of peat and other organic soils is generally lower than that of inorganic soils. It depends on the moisture content of the sample. Fig. 8 shows the relationship between dry density (γ_d) and moisture content for organic soils. As seen from the Figure, although γ_d can be as low as 0.2 g/cm³ in peaty soils, it is slightly higher and ranges between 0.7 to 1.0 g/cm³ in other organic soils.

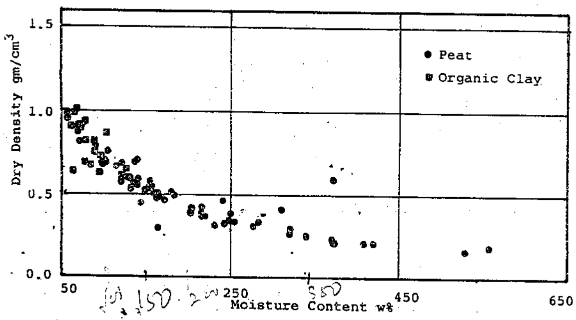


FIG 8 - RELATIONSHIP OF DRY DENSITY(γ_d) Vs MOISTURE CONTENT

Importance of strength parameters and their determination using the triaxial test were highlighted earlier. Ray et al (1986) described CU tests with pore pressure measurements on amorphous-granular peats of Sri Lanka having a natural moisture content of more than 300%. Figs. 9(a) and 9(c) shows the variation of normalised deviator stress and pore pressure with arial strain. Fig. 9(b) shows the relationship between deviator stress and mean effective stress. These results indicate that the behaviour resembles that of soft sensitive clays. Mampitiya (1987) carried out a series of triaxial tests on Colombo peats with the assistance from NBRO and the University of Moratuwa. They consisted of strain controlled, isotropically consolidated and anisotropically consolidated, drained and undrained compression tests with pore pressure measurements. Pore pressure development was found to be high compared to normally consolidated (NC) clays with the pore pressure coefficient at failure (A_f) ranging from 0.4 to 1.0. These values are slightly below the results obtained by Ray et al (1986). Pore pressure coefficient A was found to be increasing with shear strain, a clear indication of its resemblance to NC clays. Further, Mampitiya's work showed that A_f increases with consolidation pressure.

5.1 Correlation between Undrained Young's Modulus (E_u) and Undrained Shear Strength (c_u)

The accurate determination of E_u remains a difficult task for soft soils, especially organic soils. This is because of the disturbance during sampling and extrusion has a very great influence on the parameter E_u. As the disturbance does not have much effect on c_u, E_u is often related to c_u.

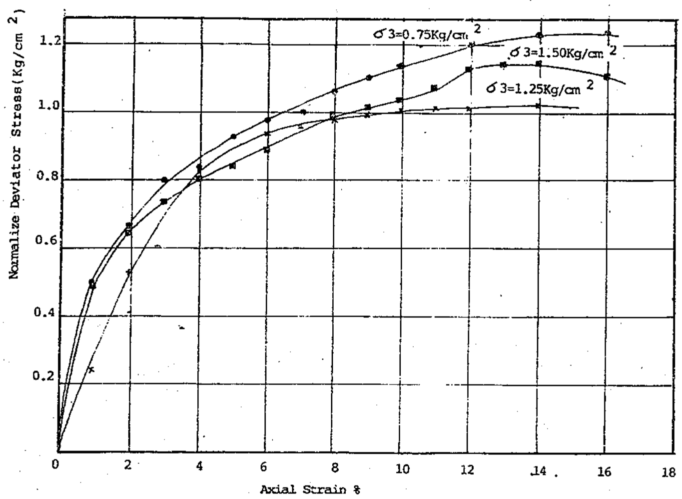


FIG 9 (a) - NORMALIZED DEVIATOR STRESS Vs AXIAL STRAIN

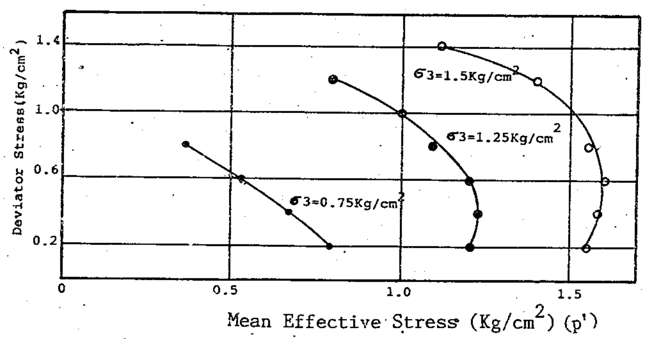


FIG 9 (b) - DEVIATOR STRESS Vs (p')

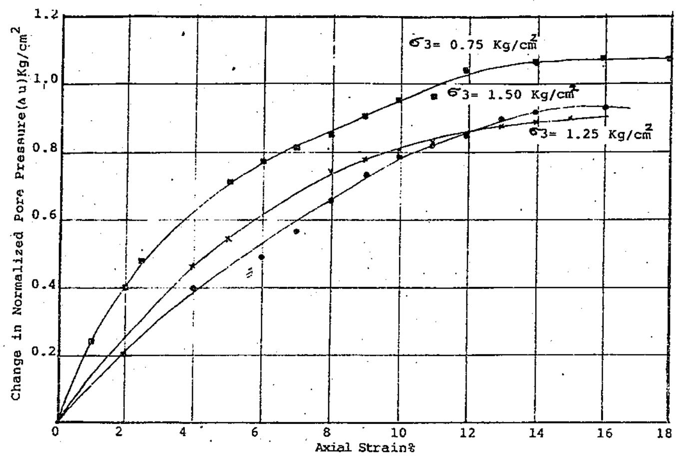


FIG 9 (c) - NORMALIZED PORE PRESSURE Vs AXIAL STRAIN

From unconfined compression tests carried out on organic soils in low lying areas, E_u/c_u ratio was found to be between 25 and 80. However, as E_u has been obtained from unconfined compression tests, this may be on the conservative side.

5.2 Correlation between Compression Index (C_c) and Insitu Void Ratio (e_o) and Liqiud Limit (w_L)

To obtain C_c a consolidation test needs to be carried out. In the absence of consolidation results one has to rely on typical values, trends or correlations. Fig. 10 shows the variation of C_c and compression ratio ($C_c/1+e_o$) for organic soils in Sri Lanka.

Correlations between C_c and e_o, and C_c and w_L had been established by many researchers in the past. Data collected from a number of sites in low lying areas in and around Colombo has made possible the following correlations (Lakshman and Sivakugan, 1987) assuming C_c is a constant during a test.

$$C_c = 0.44\, e_o - 0.16$$
$$C_c = 0.41\, w_L + 0.005$$

5.3 Correlation between Compression Index (C_c) and Recompression Index (C_r)

Recompression index is required in the settlement analysis of over consolidated clays and to predict heave in all clays. Unlike the compression index, the recompression index can be rarely correlated with e_o, w_L or Plasticity Index. From the consolidation tests carried out at NBRO on organic soils, it is found that C_r/C_c lies around 1/5 to 1/15.

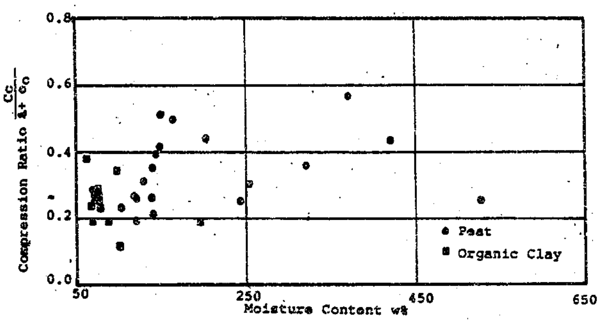


FIG 10 (a) - VARIATION OF ($C_c/1+e_o$) Vs MOISTURE CONTENT

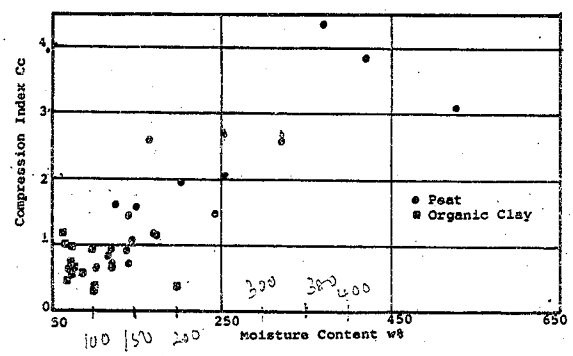


FIG 10 (b) - VARIATION OF COMPRESSION INDEX C_c Vs MOISTURE CONTENT

For simplicity, C_c is assumed to be constant for organic soils. However, observation of results of consolidation tests indicate that C_c does not remain constant for organic soils. Using results published in reputed Journals, Lakshman (1988) proposed a method of calculation of settlements of organic subsoils, not taking C_c into account, but using the organic content of the soils. He further suggested that, soils having organic contents of 20% or more (because the e V_s log p curve is nonlinear), should be considered as organic soils.

6.0 SETTING UP OF DATA BASE

Considering the importance of readily available information on subsoils in the low lying areas, NBRO has taken steps to set up a data base using personnel computer facilities available at NBRO. The following parameters have been obtained and used for the present data base.

1. Soil classification
2. Depth and thickness of each soil layer
3. Soil series
4. Settlement parameters
5. SPT N-values
6. Undrained shear strength
7. Density and specific gravity
8. Natural moisture content
9. Atterberg limits
10. Particle size distribution
11. Organic content

The data base has been developed using dBase III, Basic programming language and the Disk Operating System (DOS). For Word Processing purposes, Word Star release 4 is currently used but data importing (extracting) from other Software Packages is difficult except for Lotus 1-2-3 print files. Fig.11 gives the summary of the data base.

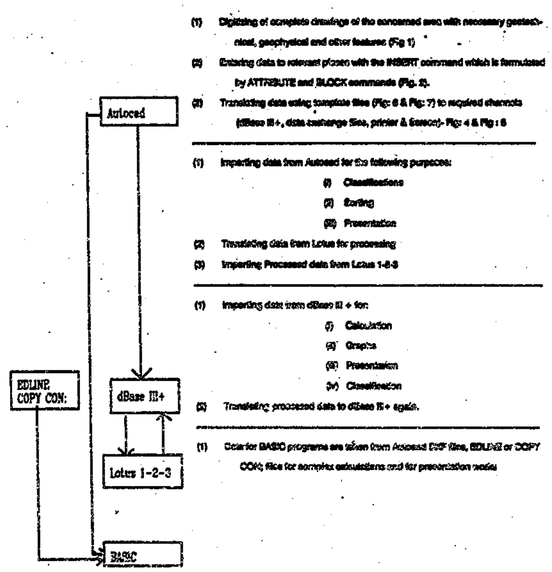


FIG 11 - SUMMARY OF THE DATA BASE

There are many advantages of a data base and should be highlighted.

1. Information on subsoil conditions of low lying areas are desperately needed by the planners and developers. When made available in a geotechnical data base, these would prove to be of immense help in preliminary and feasibility studies and decision making on optimum land use.

2. Geotechnical maps that could be prepared using this information with the aid of Autocad and Lotus computer software facilities will help the engineers in rational planning of site investigation, preliminary evaluation of possible superstructure/foundation types and in envisaging construction problems.

3. Using the data base, the engineer can decide on appropriate foundation types for a particular structure in a particular area. Such a decision at an early stage could prove to be economical in the long run.

4. A data base is useful in the storage of important data because they are easily retrievable. Very good quality presentation can be made of the data, according to requirement, by changing the template files of the Autocad data base or other files (i.e. dBase III +, Lotus 1-2-3 and BASIC Files) without much inconvenience.

5. A computer data base is flexible i.e. existing data can be modified or deleted or new data added any time. Further, parameters which are affected by changes in data can be recalculated with little time spent (e.g. when a new entry is made to Autocad file, it can be translated to dBase III + or Lotus 1-2-3 files and any recalculations may be carried out).

6. It is important to study the correlations between different parameters since they furnish valuable and important (but simple) relationships to the practicing engineer. This becomes doubly important to subsoils encountered in low lying areas. (i.e. organic soils including peat) because no in-depth or basic research has been carried out on them in Sri Lanka.

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MONITORING OF BUILDINGS IN LOW LYING AREAS

P. SAMARANAYAKE, B. VIGNESWARAN AND S. SHANMUGANATHAN

1.0 INTRODUCTION

This paper describes the programme of monitoring of buildings which was carried out with a view to obtain factual data on the performance of buildings constructed within the low lying areas. The information gathered through this programme is considered essential in realising the objectives of the research project, viz.

1) to economise and optimise foundations of structures,

2) to develop suitable foundation types and,

3) to evolve guidelines for economical design of foundations, with emphasis on low cost housing in low lying marshy areas.

The scope of this monitoring programme on buildings include the following activities;

1) measurement of settlements of buildings with time.

2) study of settlement pattern along the periphery of buildings.

3) observation and study of occurrence of distress in buildings associated with settlement and their progress.

4) study of the change of rate of settlement of buildings due to various factors and reasons, such as the rate of construction, rate of loading, environmental conditions etc.

2.0 BASIS OF SELECTION OF BUILDINGS

In order to study the performance of buildings with a view to evolve appropriate foundation types and to prepare guidelines suitable for many different conditions, it is obviously necessary to investigate a large number of buildings under numerous combinations of the following different criteria;

1) subsoil profile
2) type of foundation
3) type of superstructure
4) scale and number of storeys of the building
5) function of the building
6) existing and new buildings

2.1 Subsoil Profile

Subsoil profile plays a very important role on the performance of the buildings. For example, in certain areas, highly compressible peat will be predominant while in certain other areas less compressible organic silt or clay may be predominant. The lateral distribution, the order of existence and, the depth and thickness of the subsoil layers etc., have an important controlling effect on the response of ground to the development activities. Five different soil subseries have been identified by the National Building Research Organisation (Senanayake-1986) in the low lying areas in and around Colombo. These soil series are briefly discussed by Ameratunga et al (1989) along with the subsoil characteristics of the low lying areas that were considered in the study.

2.2 Type of Foundation

The main and the ultimate objective of the current study being the evolving of appropriate foundations, particularly for low cost housing, the study was restricted to shallow foundations because deep foundations cannot be considered economically acceptable for low cost housing under the present circumstances. There are several types of shallow foundations used in the study area which include pad, strip (both reinforced concrete and masonry), stiffened and raft foundations.

2.3 Type of Superstructure

The type of superstructure can be broadly categorised in to two, viz. structures with load bearing walls and the framed structures.

2.4 Scale and Number of Storeys of the Building

The scale of building, hence the ground area loaded and the depth of ground influenced, and the number of storeys, hence the intensity of loading, affect the ground response and therefore the performance of the building. In general, the buildings constructed in the low lying areas discussed here are mostly single or two storeyed buildings.

2.5 Function of the Building

According to the function of the building, the loads effective on the subsoils could vary largely. For instance, in a residential building, the dead load of the structure is more critical than the live load. In contrast, in a warehouse, the live load acting over a very large area could be more critical than the load of the structure.

2.6 Existing and New Buildings

For the purpose of this study, a major distinction was made between existing and new buildings. Existing buildings as the name suggests are buildings that had already been completed before any observations were made, whereas, new buildings refer to those fall in the contrary. Such a distinction was considered essential as the former type indicates the overall performance of a building several years after its construction, while in the latter type, the performance of a building can be monitored more systematically from the initial stages of development activities.

3.0 COLLECTION OF INFORMATION FOR THE SELECTION OF BUILDINGS

Several approaches were made in order to select a large number of buildings as possible using different avenues. Initially, design and construction agencies such as the Buildings Department, State Engineering Corporation, National Housing Development Authority, Urban Development Authority of the Ministry of Local Government, Housing and Construction were contacted to obtain information on completed, ongoing and proposed building projects. As the number of buildings found through the above avenue were limited, it was considered necessary to reach the agencies outside the Ministry and other relevant organizations in the private sector and individuals to gain access to many buildings for monitoring. in addition, most of the low lying areas where development has taken place or is in progress, were reconnoitered with a view to locate buildings that were suitable for monitoring. With the cooperation received from many agencies and individuals who gave consent and continued assistance to monitoring of their buildings, it was possible to select a total of 135 buildings which are classified according to zones in Table 1 and according to type of structure, no. of storeys, function and type of foundation in Tables 2,3, 4 & 5 respectively.

4.0 METHODOLOGY OF MONITORING

Any measurement should be sufficiently accurate to realise the objectives of the monitoring programme. According to Cheney J.E. (1980), for meaningful monitoring it is reasonable to record to an accuracy as good as, or better than, a tenth of the "Movement of Interest". Accuracy is defined as the closeness of approach of a measurement to the true value of the quantity measured. Accuracy is synonymous with degree of correction. On the other hand, precision means, the closeness of approach of each of a number of similar measurements to the arithmetic mean. Precision associates with reproducibility/ repeatability. Therefore although accuracy requires precision, precision does not require accuracy.

TABLE 1. CLASSIFICATION OF BUILDINGS ACCORDING TO ZONES

ZONE	NUMBER OF BUILDINGS
1. Orugodawatta	35
2. Peliyagoda	9
3. Bloemendhal	38
4. Nawala	31
5. Battaramulla	22

TABLE 2. CLASSIFICATION OF BUILDINGS ACCORDING TO TYPE OF STRUCTURES

TYPE OF STRUCTURE	NUMBER OF BUILDINGS
1. Framed Structures	91
2. Structures with Load Bearing Wall	44

TABLE 3. CLASSIFICATION OF BUILDINGS ACCORDING TO NUMBER OF STOREYS

NUMBER OF STOREYS	NUMBER OF BUILDINGS
1. Single Storey	54
2. Two Storey	72
3. Three Storey	9

TABLE 4. CLASSIFICATION OF BUILDINGS ACCORDING TO FUNCTION

FUNCTION	NUMBER OF BUILDINGS
1. Office	9
2. Warehouse	5
3. Factory	2
4. Residence	89
5. Service Station/ Car Park	4
6. Lecture Halls/ Laboratories	2
7. Museum	4
8. Bakery	

TABLE 5. CLASSIFICATION OF BUILDINGS ACCORDING TO TYPE OF FOUNDATION

TYPE OF FOUNDATION	NUMBER OF BUILDINGS
1. Masonry Strip	26
2. R/C Strip with Rubble Foundation	8
3. Masonry Strip with Plinth Beam	2
4. R/C Pad for column and R/C Strip for Wall	36
5. R/C Pad for column and R/C Strip for Wall with Plinth Beam	15
6. Vierendeel	12
7. Raft	01
8. Inverted 'T' Beam	35

4.1 Measurement of Settlement

Generally, surveying techniques are used to monitor settlements in buildings and other structures. Table 6 shows the various monitoring procedures adopted in other countries and the limitations in the accuracy of these methods.

TABLE 6 (a) MONITORING PROCEDURES AND ACCURACIES

INSTRUMENT TYPE	ACCURACY	REMARKS
1. Manometric Water Level	± 5-10 mm	Described under
2. Engineers Level and Ordinary Staff	± 5 mm	Ordinary Levelling Technique
3. Precise Level (Wild N-3) and Invar Staff	± 0.01 mm	Staff is fitted with hemispherical 'cup' foot and hold vertically with the use of a light weight stand
4. Multi Point Hydraulic Level Gauge	0.1 mm	This provides a means of recording settlement at several positions. A remote reading system provides a continuous recording capability (Ref. T.H.Hanna-Instruments for Ground Movement Measurements P.284)
5. Settlement Measuring System Using a Tilt Meter	± 0.025 mm for 1 m length of reference Beam	With settlement of the foundation, the reference beam, A, B on which the Tilt Meter was mounted, tilted and thus the settlement of B relative to A could be determined (Ref. T.H.Hanna-Instruments for Ground Movement Measurements P.317)

TABLE 6 (b) TYPES OF REFERENCE POINTS/SETTLEMENT PLUGS

TYPE	REMARKS
1. BRS Settlement Plug (Fig. 3)	This is a Stainless Steel Socket Set in a hole in the structure. A removable Plug provides a Datum for levelling.
2. Welded Steel Ball Bearings of (dia.25 mm)	Suitable for Steel Structures
3. Installation of Butt Hinges	Hinges can be folded to flush with the Wall when not in use, to avoid damage.

1) Levelling Instrument

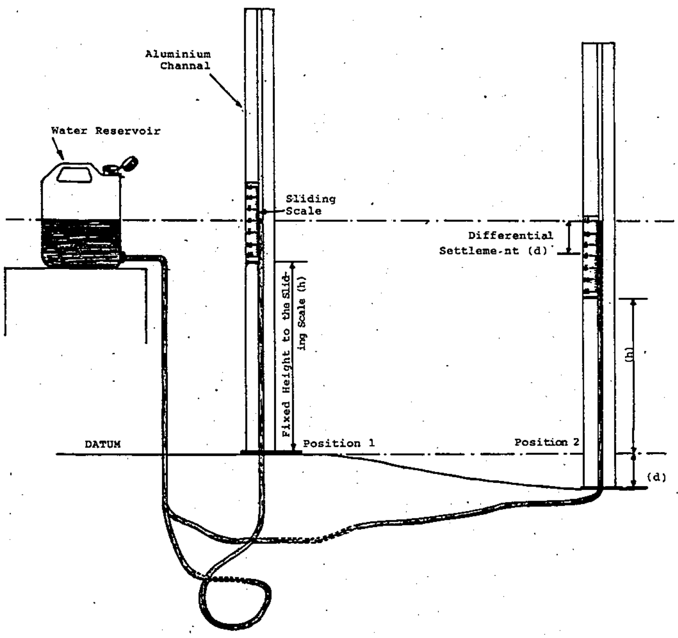
To expect a high degree of accuracy, precise leveling technique is often used. Necessary accuracy is related to three basic factors : sensitivity of the structure to settlement, the time period available for observation and purpose of the analysis. However, precise levelling is more time consuming and the equipment is more expensive. Therefore, for the work involved with this monitoring programme for the low lying areas, an ordinary survey level was used. The use of the ordinary level instead of a precision level was not considered to have an appreciable impact on the study because of the high order of settlements anticipated. However, to ensure accuracy of measurements, the initial and a few occasional intermediate readings were taken using the precision attachment.

A TOPCON 3 type Engineer's level, together with a 3.0m long staff having 5mm graduations were used to measure the settlement. However, with eye approximation, measurements could be taken to the closest 1mm. In addition to the basic equipment, a portable change point consisting of a mild steel triangular base with a small rounded plug on top was used. To minimise errors, longer sights were avoided as far as possible.

2) Manometric Water Level

For less precise work, even a water level could be used. This is sufficiently accurate and only one operator is needed. The suggestion of Building Research Establishment (BRE) is shown in Fig. 1. The water reservoir is a 2 litre capacity plastic bottle which near its bottom is connected to a flexible tube about 10m long. The other end of the tube which is used for observation purposes, runs through a light weight aluminium section, and therefore can be held vertical without difficulty. A sliding scale on the aluminium section acts as the levelling staff for observations. When the staff base is held at the chosen datum and after the dyed water in the tube has come to rest and found its own level (ie. the water level in the reservoir), the mid-point zero of the sliding scale on the staff is locked against the water meniscus. Without disturbing the bottle or the scale, the staff is then read again against the water meniscus at successive positions to be monitored along the course.

If the flexible tube is not long enough, the bottle must be re-positioned, while the staff is kept stationary. Immediately after repositioning, the scale should be re-set. Just as in levelling, the course should be completed and closed at the original datum. A closing error of ± 5 mm is considered satisfactory in general. However, this could increase up to $\pm$ 10mm when several change points are used.



MANOMETRIC WATER LEVEL

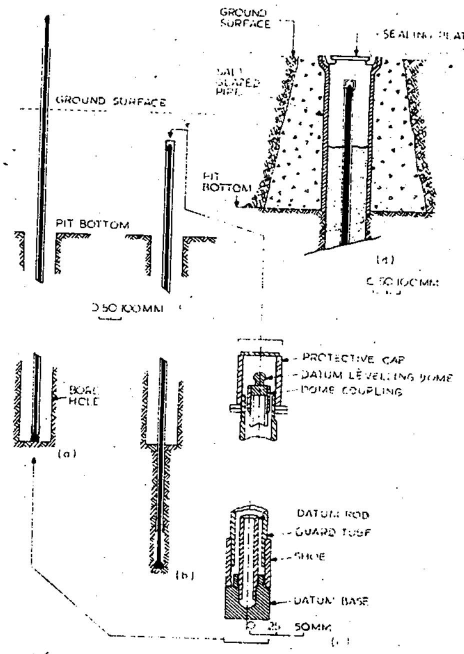
FIG - 1

3) Bench Marks

A pile or any structure, directly or through piles, firmly supported on bed rock is considered as a desirable bench mark for monitoring settlements in buildings in the low lying areas (see Fig. 2 for typical bench mark). However, in most instances, it is not possible to locate a suitable firmly supported structure or to install a costly permanent bench mark, eg. a pile, in the vicinity, exclusively for this purpose. In such cases, old structures existing in the neighbourhood, which can be considered as stable or undergoing only very negligible settlement over the period of monitoring, were used as temporary bench marks. To check reliability, a few additional points are established. The stability of such bench marks were checked occasionally in relation to the nearest permanent bench mark to ensure better accuracy.

4) Settlement Plugs

To monitor settlements using a level, number of measurement points have to be installed on the periphery of the buildings. Levelling station should be built into the building and designed to enable accurate levelling. They should not be liable to accidental damage or damage by vandals. But most importantly, it should be acceptable to the architect.

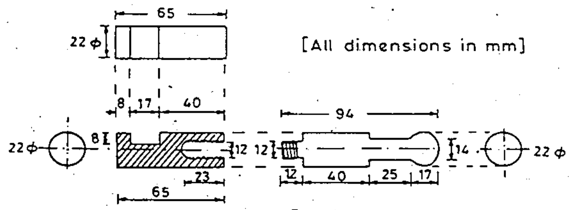


BRS 6 m datum (a) before driving, (b) installed position, (c) component detail, (d) site protection for datum (CHENEY, 1974)

Fig.2.Typical Bench Mark

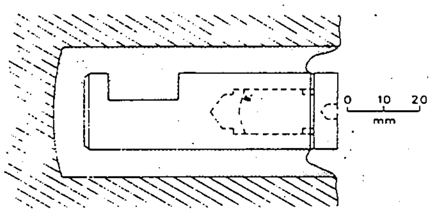
BRS levelling station as described by Cheney (1973) (see Fig. 3) consists of a stainless steel socket, 65 mm long and 22 mm diameter, which is set in a hole in the building, and a reasonable levelling plug of about the same dimensions on which the base of a staff is held when levels are taken. When not in use, the socket is sealed from dust by a protective Persfex bung. The face of the bung is flush with the structural finish and the Persfex takes on the colour of the finish, making it barely visible. This is designed to ensure that the plug will position with repeatable accuracy. The thread is a loose fit and is used only to pull the plug into the socket. The socket needs only to be set approximately horizontal, since the ball-shaped end of the plug on which the staff rests, will accommodates considerable off-level installation of the socket.

Another simple hinge device is used, where the arm can be rotated to horizontal position for taking readings and closed in flush with the wall once the readings are taken.

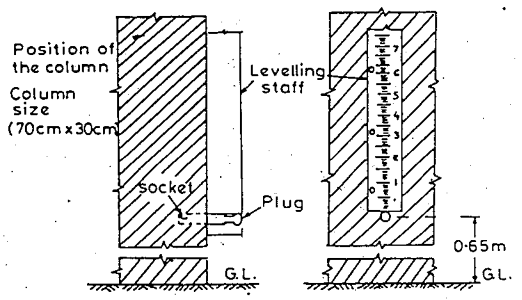


Longitudinal section of the socket

Settlement plug



Section of a BRS levelling station grouted in a prepared hole



BRS LEVELLING STATION
FIG - 3

At NBRO, considering the high cost of such plugs and their greater susceptibility to vandalism, two simple types of plugs were used. Either durable ceramic tiles of about 25mm x 50mm or steel hooks with a rounded edge were installed on the periphery of the building (Fig. 4). Ceramic tiles were placed on flat surfaces such as plinth beams etc. and steel hooks were installed in walls and columns.

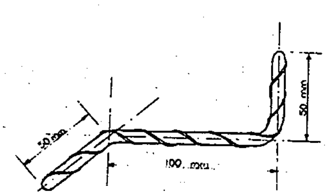
5) Frequency of Readings

The frequency of readings should be related to the rate of construction and to the rate at which the readings are changing. While too many readings will unnecessarily prolong processing, too few may cause important events to be missed.

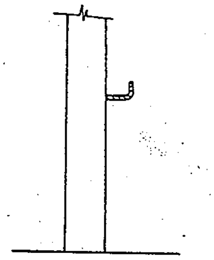
Several initial readings are generally required to establish a reliable base. During construction time, readings are taken once a week or at least once in two weeks. Additional readings are taken when it is considered important and necessary, for example after a heavy rain or when a sudden load is added.

4.2 Measurement of Tilt

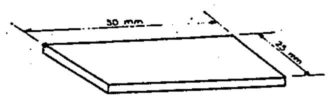
The tilt of a building occurs due to differential settlements. In most buildings, tilting is hardly noticeable, especially when the construction errors are prominent.



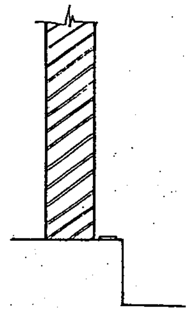
(a) Hook made of Reinforcement Bar



Hook Embeded into a Column



(b) Ceramic Plate



Ceramic Plate Bedded in Cement Mortar

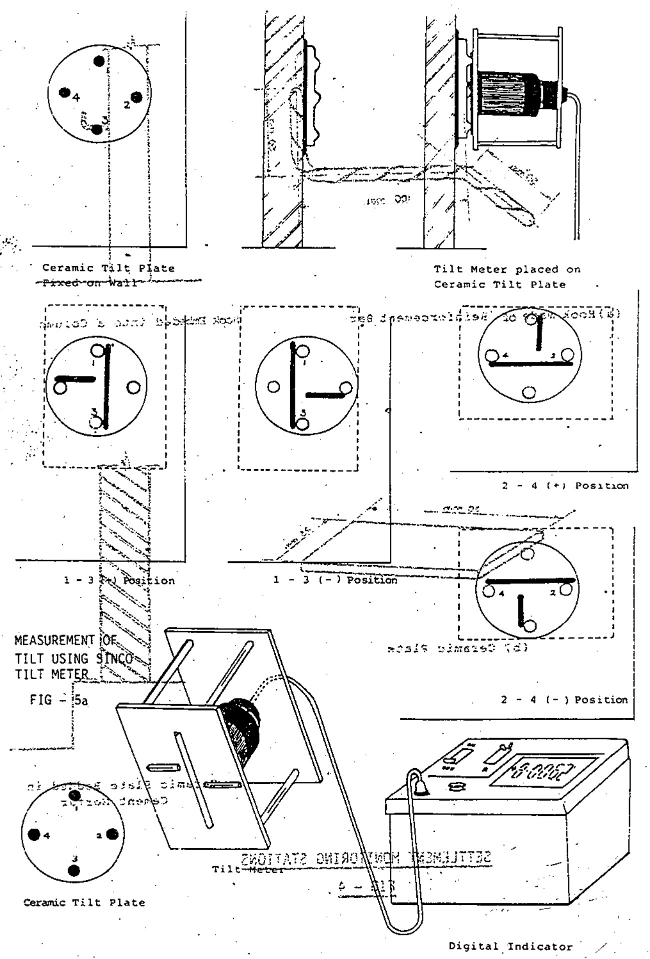
SETTLEMENT MONITORING STATIONS
FIG - 4

1) Tiltmeter

When the magnitude or the rate of tilt is very small, the best method of observation is to use a sensitive tiltmeter. The SINCO tiltmeter used at NBRO consists of three units, viz. a portable tiltmeter sensor, a portable digital indicator and a ceramic tiltplate. The ceramic tiltplates are permanently attached to the structure (or wall) whose tilt is under observation, using an adhesive or cement. When the sensor is placed on the ceramic plate (which has four protruding pegs) the angle of tilt is shown in the digital indicator [(Fig 5 (a)]. The smallest change in the angle of tilt that can be detected is approximately 20 seconds. An inclinometer could also be used if necessary for the same purpose. A more simple one that could be made locally is shown in Fig. 5 (b).

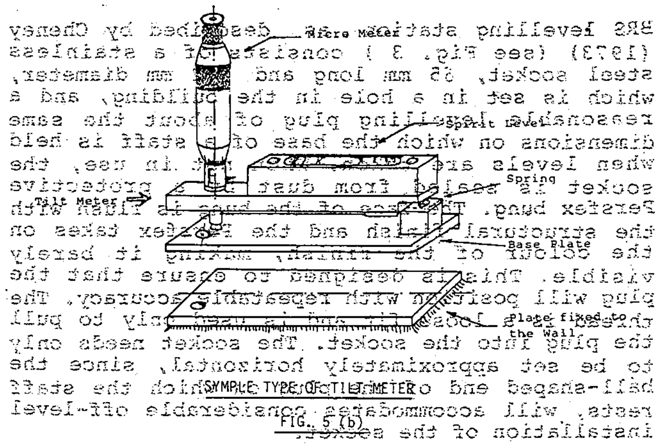
2) Theodolite and Plumbline

A theodolite can also be used to measure the tilt of a wall if access is available beyond and in line with the length of the wall. However, the most economical method would be to use a plumb bob and a ruler for low-rise buildings. Plumbline is held against near the top of the wall and when the bob hangs freely and stationary, the distance to the plumbline from the wall is measured using a ruler. BRE suggests the bob to be immersed in water to damp oscillations.



MEASUREMENT OF TILT USING SINCO TILT METER

FIG - 5a



SIMPLE TYPE OF TILT METER

FIG. 5 (b)

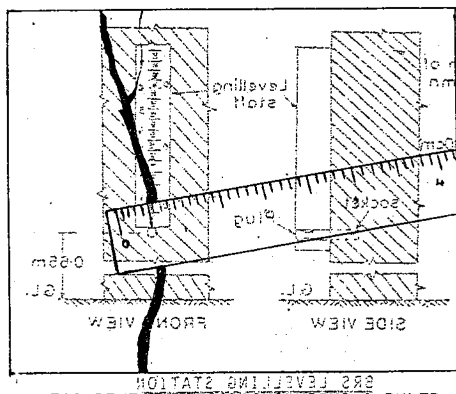
4.3 Monitoring of Cracks

The main form of distress in buildings is cracks in structural members. Although, cracks could appear in buildings due to various reasons, the interest here is directed to cracks due to settlements only.

In monitoring cracks, it is very important to record the date of the initial observation of the crack. Moreover, it is desirable to monitor the progress of the crack, ie. increase in length and width. Generally, the position and sizes of cracks need to be recorded in a way sufficient only to determine the scale and extent of the damage.

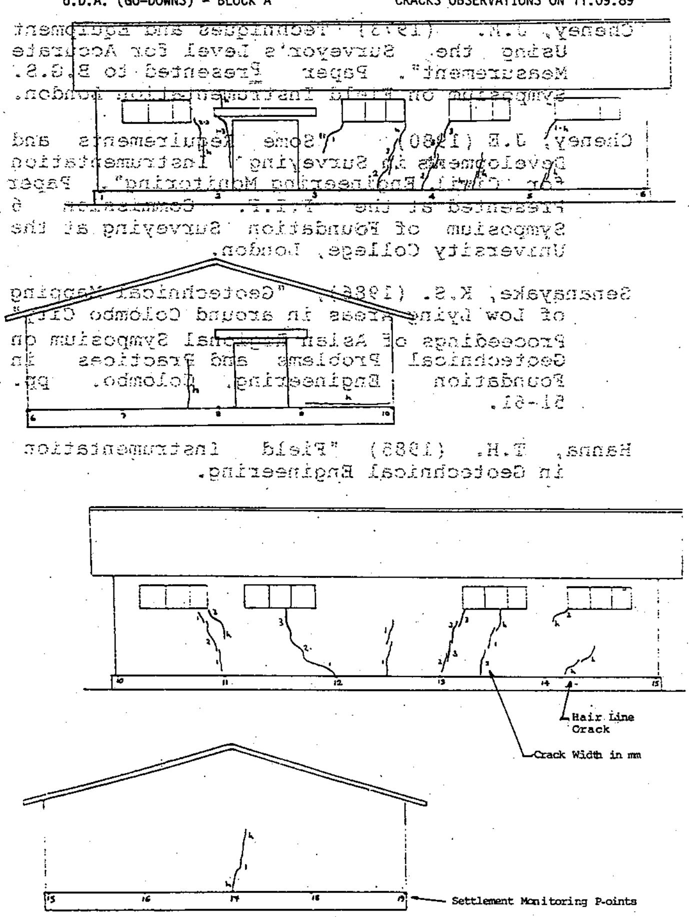
In monitoring the performance of buildings, the accuracy required for crack measurement is not more than 1mm. For smaller cracks, either 1/2mm or "hairline" designations may be used. Accuracy beyond the above is hardly necessary. On the other hand, due to the nature of cracks and the condition of wall surface etc., it may not be possible to achieve a very high level of accuracy.

The width of crack can be easily measured using a graduated ruler held on the surface at right angles to the crack (Fig. 6). To record the extent of cracks, it is desirable to draw free hand sketches, on a scaled drawing of the elevation of walls. A typical sketch is shown in Fig. 7.



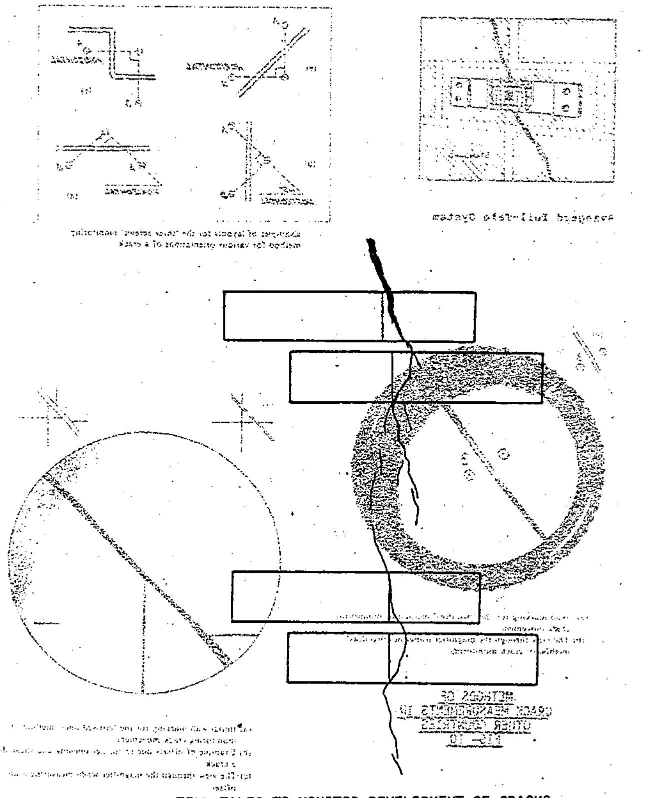
MEASUREMENT OF CRACKS USING A RULER

FIG 6



RECORDING OF OBSERVED CRACKS

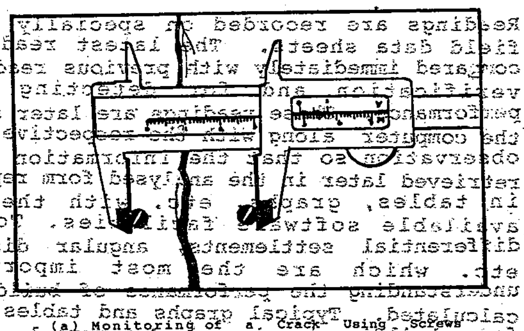
FIG. 7



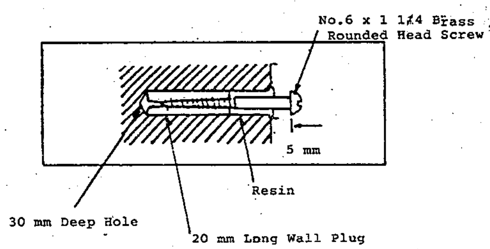
TELL-TALES TO MONITOR DEVELOPMENT OF CRACKS

FIG - 8

The most common method that had been adopted in Sri Lanka to monitor cracks is the use of glass tell-tales stuck across the crack. An improvement of this method is the use of two glass plates, one on each side of the crack. By having the cursors on the overlapping plates (Fig. 8) initially aligned, these tell-tales can be used to monitor changes in crack width. While these are very simple methods, even any owner or occupant of a building could use, for more accurate measurements, screws and calipers could be used. Two holes are drilled a few centimetres apart across the crack, and two small screws are firmly set in the wall. Using a caliper gauge, the distance between the screws and therefore any changes in the width of cracks could be monitored (Fig. 9). Photographs of the crack taken from time to time with a micro-scale placed by the side for calibration are useful in more accurate measurements though very expensive and not required in the present work. Fig. 10 shows further methods of crack measurements which are adopted in other countries.



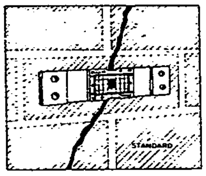
(a) Monitoring of a Crack Using Screws and a Caliper



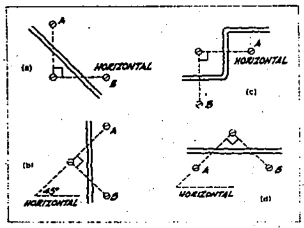
(b) A Screw Set with Wall Plug and Resin Adhesive

SCREW AND CALIPER METHOD OF CRACK MEASUREMENT

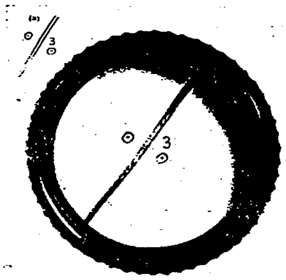
FIG - 9



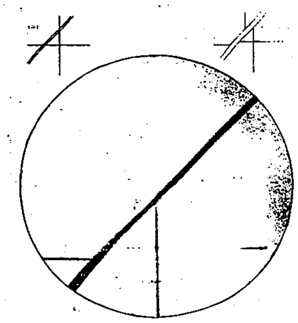
Avangard Tell-Tale System



Examples of layouts for the 'three screws' monitoring method for various orientations of a crack



(a) Wall marking for the 'two dots' method of monitoring crack movement
(b) The view through the magnifier using the 'two dots' method of crack monitoring



(a) Initial wall marking for the 'crossed lines' method of monitoring crack movement
(b) Example of offsets due to further opening and shear in a crack
(c) The view through the magnifier when measuring a new offset

METHODS OF CRACK MEASUREMENTS IN OTHER COUNTRIES FIG- 10

5.0 DOCUMENTATION AND PRESENTATION OF RESULTS OF MONITORING

Readings are recorded on specially prepared field data sheets. The latest readings are compared immediately with previous readings for verification and for detecting unusual performance. These readings are later stored in the computer along with the respective dates of observation so that the information could be retrieved later in the analysed form represented in tables, graphs, etc. with the aid of available software facilities. Total and differential settlements, angular distortion, etc. which are the most important in understanding the performance of buildings are calculated. Typical graphs and tables obtained for a building is given in Annexure 1.

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ANNEXURE 1

LOW LYING AREA RESEARCH PROJECT
SITE :U.D.A (GO DOWNS)
BLOCK : A

SETTLEMENT IN MILLIMETRES

No. OF DAYS / DATE		30-Oct-86	13-Nov-86	26-Nov-86	11-Dec-86	25-Dec-86	11-Jan-87	01-Feb-87	02-Mar-87	06-Apr-87	21-May-87	24-Jun-87	20-Aug-87
		0	14	27	42	56	73	94	123	158	203	237	294
LOCATION	1	0	-3	-9	-10	-13	-14	-14	-16	-19	-21	-21	-24
	2	0	-6	-10	-10	-13	-15	-15	-16	-18	-18	-18	-23
	3	0	-4	-8	-8	-11	-13	-13	-13	-17	-17	-17	-20
	4	0	-3	-6	-7	-9	-11	-11	-13	-18	-18	-18	-22
	5	0	-4	-5	-7	-8	-11	-12	-15	-20	-20	-21	-26
	6	0	-4	-4	-6	-8	-11	-13	-15	-21	-21	-25	-28
	7	0	-3	-4	-6	-8	-10	-10	-14	-18	-19	-22	-26
	8	0	-3	-5	-6	-8	-9	-9	-14	-16	-17	-20	-23
	9	0	-2	-4	-6	-7	-8	-8	-12	-14	-14	-17	-20
	10	0	-2	-4	-5	-6	-7	-7	-10	-13	-13	-13	-17
	11	0	-2	-4	-5	-7	-7	-7	-11	-13	-13	-16	-19
	12	0	-2	-2	-5	-7	-8	-8	-12	-15	-17	-18	-23
	13	0	-4	-4	-7	-9	-10	-11	-15	-20	-23	-23	-28
	14	0	-6	-8	-11	-14	-15	-17	-22	-28	-32	-35	-37
	15	0	-7	-7	-10	-13	-14	-16	-24	-31	-36	-37	-40
	16	0	-7	-11	-12	-14	-17	-18	-24	-30	-34	-35	-39
	17	0	-5	-10	-11	-13	-16	-17	-21	-25	-29	-30	-33
	18	0	-4	-10	-11	-13	-15	-16	-19	-23	-25	-26	-29
	1	0	-3	-9	-10	-13	-14	-14	-16	-19	-21	-21	-24

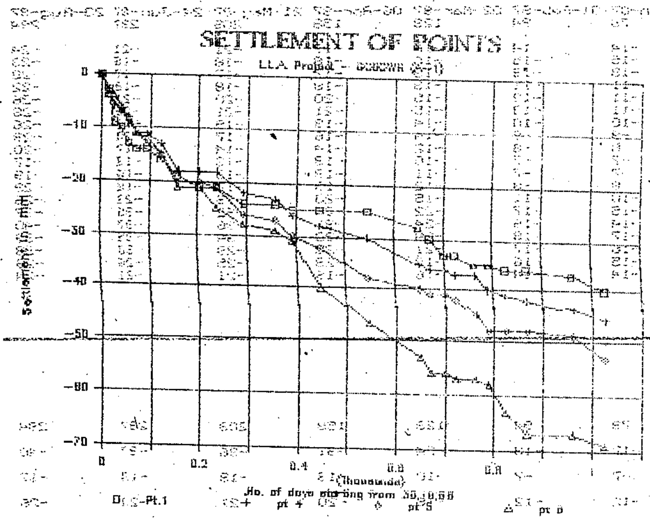
SITE :U.D.A (GO DOWNS)
BLOCK : A

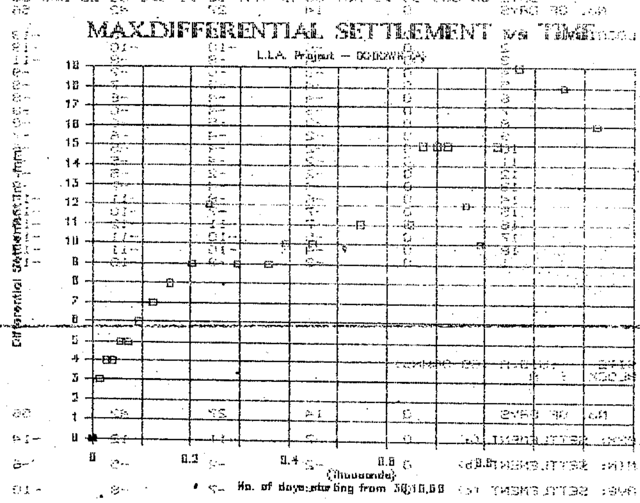
No. OF DAYS	0	14	27	42	56	73	94	123	158	203	237	294
MAX: SETTLEMENT (a)	0	-7	-11	-12	-14	-17	-18	-24	-31	-36	-37	-40
MIN: SETTLEMENT (b)	0	-2	-2	-5	-6	-7	-7	-10	-13	-13	-13	-17
AVG: SETTLEMENT (c)	0	-4	-7	-8	-10	-12	-12	-16	-20	-21	-23	-26
MAX: DIF: SETTL (d)	0	3	4	4	5	5	6	7	8	9	12	9
INV.MAX.ANG.DIS (e)		2033.3	1237.5	1525.0	1220.0	1220.0	1016.7	871.4	762.5	677.8	508.3	677.8
L =(a)/(a-b)		1.4	1.2	1.7	1.7	1.7	1.6	1.7	1.7	1.6	1.5	1.7
M =(a)*(e)/25.4		560.4	535.9	720.5	672.4	816.5	720.5	823.4	930.6	960.6	740.5	1067.4
N =(a-b)*(e)/25.4		400.3	438.5	420.3	384.3	480.3	440.3	480.3	540.4	613.7	480.3	613.7

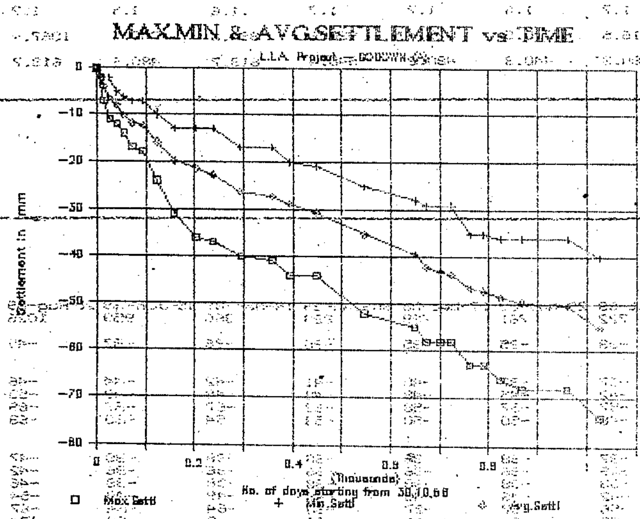
SETTLEMENT IN MILLIMETRES

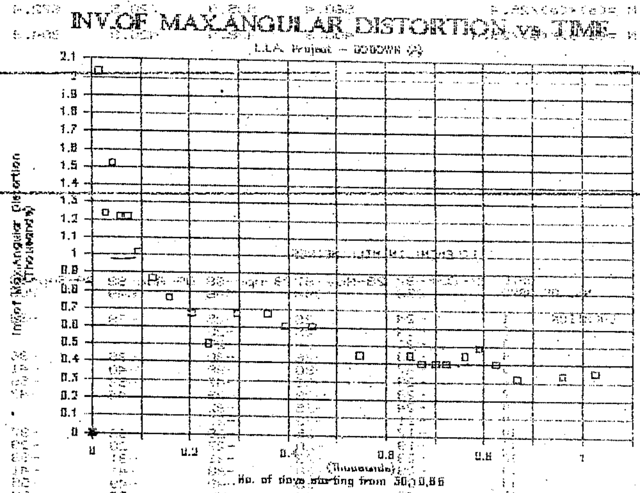
No. OF DAYS / DATE		23-Oct-87	28-Nov-87	28-Apr-88	09-Aug-88	29-Sep-88	21-Oct-88	29-Nov-88	27-Dec-88	30-Jan-89	14-Mar-89	15-Jun-89	21-Aug-89
		358	394	546	649	700	722	761	789	823	866	959	1026
LOCATION	1	-24	-25	-25	-28	-33	-33	-35	-35	-36	-36	-37	-40
	2	-24	-25	-26									
	3	-21	-24	-27									
	4	-23	-26	-30	-35	-36	-37	-37	-40	-41	-42	-44	-46
	5	-27	-30	-38	-40	-41	-42	-45	-48	-48	-48	-49	-53
	6	-29	-31	-47	-53	-56	-57	-57	-58	-63	-67	-67	-69
	7	-28	-28	-39	-45	-47	-48	-50	-50	-53	-54	-60	-63
	8	-24	-35	-32									
	9	-21	-22	-27									
	10	-17	-25	-28	-33	-37	-37	-38	-38	-38	-38	-38	-47
	11	-19	-20	-27	-28	-29	-29	-36	-36	-36	-36	-36	-42
	12	-23	-23	-31	-31	-33	-33	-38	-38	-38	-38	-38	-46
	13	-28	-38	-38	-41	-41	-42	-45	-48	-48	-49	-49	-54
	14	-37	-38	-49	-49	-55	-56	-57	-57	-60	-64	-64	-67
	15	-40	-44	-50	-55	-58	-58	-61	-61	-56	-68	-68	-74
	16	-41	-43	-52	-49	-58	-58	-63	-63	-63	-64	-64	-72
	17	-36	-36	-43	-43	-48	-51	-53	-53	-55	-56	-56	-58
	18	-31	-31	-32	-32	-42	-42	-46	-45	-47	-47	-48	-52
	1	-24	-25	-25	-28	-33	-33	-35	-35	-36	-36	-37	-40

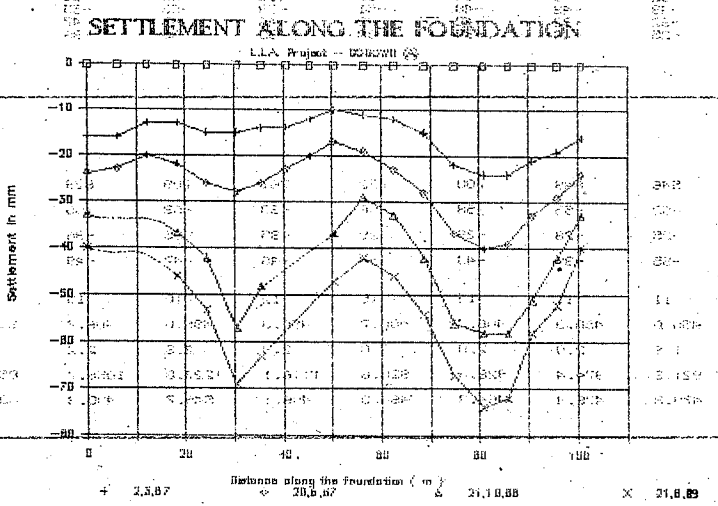
No. OF DAYS	358	394	546	649	700	722	761	789	823	866	959	1026
MAX: SETTLEMENT	-41	-44	-52	-55	-58	-58	-63	-63	-66	-68	-68	-74
MIN: SETTLEMENT	-17	-20	-25	-28	-29	-29	-35	-35	-36	-36	-36	-40
AVG: SETTLEMENT	-27	-29	-35	-39	-43	-44	-46	-47	-49	-50	-50	-55
MAX: DIF: SETTL	9	10	11	11	15	15	12	10	15	19	18	16
INV.MAX.ANG.DIS	677.8	610.0	450.0	450.0	406.7	406.7	450.0	495.0	406.7	321.1	338.9	353.6
L =(a)/(a-b)	1.7	1.8	1.9	2.0	2.0	2.0	2.3	2.3	2.2	2.1	2.1	2.2
M =(a)*(e)/25.4	1094.1	1056.7	921.3	974.4	928.6	928.6	1116.1	1227.8	1056.7	859.5	907.3	1030.1
N =(a-b)*(e)/25.4	640.4	576.4	478.3	478.3	464.3	464.3	496.1	545.7	480.3	404.5	426.9	473.3


SETTLEMENT OF POINTS
L.I.A. Project -- GODOWN (A)
Settlement (in mm)
No. of days starting from 30,10,88
(Thousands)
Pt.1
pt 4
pt 5
pt 6


MAX.DIFFERENTIAL SETTLEMENT vs TIME
L.I.A. Project -- GODOWN (A)
Differential Settlement (in mm)
(Thousands)
No. of days starting from 30,10,88


MAX.MIN.& AVG.SETTLEMENT vs TIME
L.I.A. Project -- GODOWN (A)
Settlement in mm
(Thousands)
No. of days starting from 30,10,88
Max.Settl
Mn.Settl
Avg.Settl


INV.OF MAX.ANGULAR DISTORTION vs TIME
L.I.A. Project -- GODOWN (A)
Inv.of Max.Angular distortion (Thousands)
(Thousands)
No. of days starting from 30,10,88


SETTLEMENT ALONG THE FOUNDATION
L.I.A. Project -- GODOWN (A)
Settlement in mm
Distance along the foundation (m)
2,3,87
20,6,87
21,10,88
21,8,89

ANALYSIS OF BEHAVIOUR OF BUILDINGS

N. SIVAKUGAN, T. SRITHARAN AND J.J.P. AMERATUNGA

1.0 SETTLEMENT

Settlement of a structure consists generally of three components namely:

1. Immediate settlement
2. Consolidation settlement
3. Secondary settlement.

Any one of the above components could be predominant depending on the soil type.

The immediate settlement, as the name implies, is the component of settlement which occurs immediately upon loading. This is an elastic distortion at undrained state in cohesive soils and occurs at constant volume. Immediate settlement is significant in cohesionless soils where it occurs due to instantaneous dissipation of pore pressure.

Consolidation settlement is a time dependent deformation where the excess pore pressure dissipate with time, and the effective stress increases. The applied load, immediately carried fully by the pore water, is gradually transferred to the soil skeleton. Expulsion of water from the soil during consolidation, results in settlement, which gradually increases with time. It occurs in all soils and most significant in saturated fine grained soils. In coarse-grained soils it occurs almost instantaneously with load application and is treated as immediate settlement.

Secondary settlement is due to creep or viscous flow which would not affect the pore pressure or effective stress. It is also a time dependent deformation which is assumed to take place at constant effective stress. Unless a soil exhibits substantial creep behaviour (e.g. peat) secondary settlement is often ignored.

2.0 RELATIONSHIP BETWEEN DISTRESS AND SETTLEMENT

A properly designed foundation should not settle excessively under the working load. The settlement is not only aesthetically undesirable, is known to initiate distress of the superstructure. The total settlement implies the downward displacement of a point in the building. If the movement is upwards, it is termed heave. However, the stability of a structure is generally not governed by the total settlement but differential settlement between two parts of a structure. Therefore, it is important to know various movements a structure can undergo (see Tennekoon,1989).

The settlement, as it was defined, is a continuous process, producing settlements of different magnitudes throughout the building over a very long period. Nevertheless, after some time, generally the rate of settlement decreases to insignificant levels.

It was mentioned that at a given time, settlement can be quantified in terms of total and differential settlements, and distortion. Excessive total settlement does not usually cause any structural distress. They affect the grading of the floor and thus the drainage, service lines such as water, sewer or electricity lines, and the aesthetics of the building. It is the differential settlement that causes structural damage and this needs to be controlled. The reasons for differential settlement can be numerous;

a) Presence of pockets or lenses of soils with different compressibilities,

b) Non-horizontal interfaces between layers, i.e. varying thickness of soil layers in the lateral extent,

c) Non-uniform loading over the building area. e.g. presence of lift wells, storage rooms etc.,

d) Time lags in the construction of adjacent parts of a structure,

e) Large loaded areas on flexible foundations.

The other causes of settlement in general are:

a) Ground water lowering. e.g. dewatering, seasonal lowering of water table,

b) Vibration or shock due to pile driving, blasting etc.,

c) Seasonal shrinkage of expansive clays,

d) Adjacent excavation, underground erosions etc.,

e) Collapse of underground cavities such as sink holes,

f) Chemical action including decay, decomposition of organic material,

g) Lack of lateral support in adjacent excavation.

2.1 Design Criteria

The design of foundations is generally based on the following criteria.

1) the actual bearing stress should not be greater than the mobilised resistance of the soil, i.e. the bearing capacity

2) estimated settlement under the bearing capacity should not exceed a tolerable value.

The first criterion is generally satisfied for buildings constructed over reclaimed land in low lying areas. As the differential settlement is related to distress and not total settlement, the second criterion requires a limit on differential settlement that can be allowed. However, because the estimation of differential settlement is difficult and time consuming, the general practice is to limit the total settlement thereby automatically limiting the differential settlement. Terzaghi and Peck (1967) recommended an allowable settlement of one inch for shallow foundations on sand assuming that the differential settlement will not be more than 75% of the total settlement. Table 1 gives limits proposed by several other authors.

Table 1:

Maximum Permissible Total Settlement for Buildings

Type of Foundation	Maximum Permissible Total Settlement (mm) Sand	Clay	Source
1.Footings of masonry walled structres	75-100*		USSR Bldg Code-1955
	25-50*		Sowers/Sowers -1961
	25	50	Nayak -1977
2.Isolated Foundations of framed structures	100*		USSR Bldg Code-1955
	40-	65	IS 1904 -1961
	50-100*		Sowers/Sowers -1961
	40	65	Nayak -1977
	50	75	Grant et al -1974
	24	30	Matsuura -1976
	38	63	Skempton et al-1956
3.Raft Foundations	40-65	65-100	IS 1904 -1961
	50-75	100-200	Nayak -1977
	65	100	Grant et al -1974
	30	40	Matsuura -1976

(*) recommended value independent of soil type.

2.2 Appearance and Classification of Cracks

Damage in buildings usually starts by cracking of partitions, panels, floor, plaster etc., which is called architectural damage. The cracking of structural elements such as beams, columns, or foundations is referred to as structural damage.

Polshin and Tokar (1957) said that the cracks become visible only when the critical tensile strain, e_{crit} is exceeded. For brickwork, hollow-tile block, and clinker block in-fills, e_{crit} is about 0.01 to 0.05% (Polshin and Tokar, 1957). For concrete, e_{crit} is about 0.02 to 0.05% (Burland and Worth, 1974). Damage classification according to BRE with the use of the width of crack is given in Table 2.

Table 2

CLASSIFICATION OF VISIBLE DAMAGE TO WALLS WITH PARTICULAR REFERENCE TO EASE OF REPAIR OF PLASTER AND BRICKWORK OR MASONRY

Category of damage	Degree of damage	Description of typical damage	Approximate crack width mm
0	Negligible	Hairline cracks of less than 0.1 mm width are classed as negligible	Up to 0.1$^{(2)}$
1	Very slight	Fine cracks which can be treated during normal decoration Perhaps isolated slight fracturing in building. Cracks rarely visible in external brickwork.	Up to 1$^{(2)}$
2	Slight	Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks not necessarily visible externally:some external repointing may be required to ensure weathertightness. Doors and windows may stick slightly.	Up to 5$^{(2)}$
3	Moderate	The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5 to 15$^{(2)}$ for a number of cracks up to 3)
4	Servere	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distorted, floor slopting noticeably walls leening$^{(3)}$ or bulging noticeably, some loss of bearing in beams. Service pipes distrupted	15 to 25$^{(2)}$ but also depends on number of cracks
5	Very severe	This requires a major repair job involving partial or complete re-building. Beams lose bearing, wall lean badly and require shoring. Windows broken with distortion. Danger of instability.	usually greater than 25$^{(2)}$ but depends on number of cracks.

Notes : 1. It must be emphasised that in assessing the degree of damage account must be taken of the location in the building or structure where it occurs and also of the function of the building or structure.
2. Crack width is one factor in assessing category of damage and shold not be used on its own as direct measure of it.
3. Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.

3.0 ELASTIC STRESS DISTRIBUTION

For accurate settlement prediction, knowledge of the initial and subsequent stresses due to loading, is essential. In the conventional practice, the load distribution within a building is calculated assuming the base of the building to be fixed.

To consider the vertical stress distribution with depth beneath the loaded area, the contact pressure has to be found. The contact pressure is calculated assuming the base to be rigid. This ensures a linear distribution of pressure beneath the footing and for a symmetrical footing this yields a uniform stress distribution.

In a homogeneous subsoil profile, assuming linear elastic soil behaviour, the increased stress on an element of soil, at a depth in the strata below the foundation can be calculated using either Boussinesq or Westergaard theories.

A simple approximation to obtain stress increment, popular among the engineers, is the 2:1 method. This is a good approximation of the average stress beneath a foundation as computed by the Boussinesq method. It assumes the load Q at the foundation to be supported on a truncated pyramid of soil having outward side slopes of 1:2 as shown in Fig.1 . Therefore, the stress increment ($\Delta\sigma_z$) at a depth Z is given by,

$$\Delta\sigma_z = Q/(L+Z)/(B+Z)$$

where B and L are dimensions of the footing. For a strip footing, the above equation reduces to,

$$\Delta\sigma_z = qB/\ (B+Z)$$

where q is the load intensity.

In multi-layer subsoil profiles, the above methods are useful in determining the stresses acting on the particular layer.

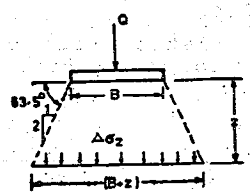


Fig.1

4.0 PREDICTION OF SETTLEMENT

Ideally, if the loadings, stratifications of the soil profile, and the soil characteristics are known precisely the settlement analysis is rather straight forward. A realistic settlement prediction requires the following three:

i) Soil model and methodology for settlement analysis. The soil model should be simple, practical, and model the soil behaviour satisfactorily at all stages of loading and at all times,

ii) Good knowledge of the loading conditions and an accurate representation of this in the soil model,

iii) Good estimates of required soil parameters. Very often the economic considerations do not permit a thorough geotechnical investigation.

In inorganic clays, consolidation settlements constitute major fraction of the settlement with very small contributions from immediate and secondary settlement. This is not the case with organic clays and peats, where all three components, immediate, consolidation, and secondary settlements contribute significantly.

4.1 Calculation of Immediate Settlement

The immediate settlement is caused by the change in shape of the soil due to increase in stress. With the assumption of elastic behaviour of soil, the immediate settlement can be computed. The Boussinesq analysis for deflection integrated for a loaded area within a uniform pressure of q_{net} results in the expression.

$$\rho_i = C_d\, q_{net}\, B\ (1-\mu^2)/E$$

where,
ρ_i = immediate settlement
q_{net} = net stress at foundation level
B = width of footing
μ = Poisson's ratio
E = Young's modulus
C_d = influence factor

C_d depends on the following;

1) foundation dimensions,

2) where the settlement is required (e.g. centre, corner or average),

3) stratification.

When the subsoil is uniform to infinite depth, the influence factors can be found from tables provided in standard handbooks on Foundation Engineering. However, when a rigid stratum is at a limited depth different influence factors have to be used (see Bowles, 1982).

Since immediate settlement occurs under undrained conditions in saturated clays and similar soils, Poisson's ratio = 0.5 is generally used. However for sandy soils, the value of Poisson's ratio is low. The Poisson's ratio is not very sensitive to values of E. Even when it is zero, the settlement will be only 25% more. Table 3 gives typical values for the Poisson's ratio.

The Young's modulus also depends on density, stress history and stratification of the soil mass. In cohesive soils, an undrained modulus, E_u should be used while in free draining soils, a drained modulus E' should be used. Young's modulus can be determined either from laboratory tests or field tests (Ameratunga et al, 1989).

Table 3 : Typical values for Poisson's Ratio

Soil Type	*Poisson's Ratio*
Clay, saturated	0.50
Clay with sand and silt	0.30—0.42
Clay, unsaturated	0.35—0.40
Loess	0.44
Silt	0.30—0.35
Sandy soil	0.15—0.25
Sand	0.30—0.35
Rock	0.10—0.40

In a layered medium, it is necessary to find an appropriate E value for calculation purposes. The most convenient method is to average the value of E of each layer to the depth of influence of the foundation (The depth of influence of the foundation for settlement purposes is generally taken around twice the width of the foundation). This could be better estimated by averaging E within the depth of influence, with the layer thickness taken into account. Fraser and Wardle (1976) suggest an approximate graphical method using weighted modulus of each layer which is described by Tennekoon (1989) also giving typical E values for soils.

4.1.1 Depth Correction Factor

Footings are generally placed below the ground surface (say at a depth, D) and at least 30cm in the case of housing. Fox (1948) proposed a correction to be applied to the surface deflection which is a function of Poisson's ratio and the depth of embedment. Bowles (1982) programmed the equations of Fox and obtained a simple presentation of depth factor against D/B (where B is the breadth of foundation) for different footing lengths and Poisson's ratios and is shown in Fig.2. Although this correction was initially proposed to correct the predicted immediate settlements only, it is now being used to correct consolidation settlements (Tomlinson, 1975).

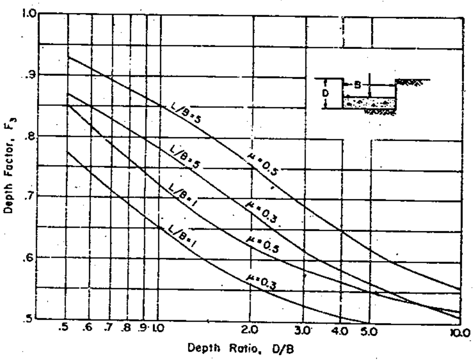


Fig.2. Depth Correction Factor

4.2 Estimation of Consolidation Settlement

Consolidation settlement refers to settlement due to primary consolidation. The rate of settlement is controlled by the rate of water being expelled from the voids in the soil mass. In coarse grained soils, it occurs instantaneously and is treated as immediate settlement.

Consolidation settlement is considered as one dimensional for settlement calculations. This is because the use of Terzaghi's equation is valid for 1-D situation only. Such an assumption is reasonable only when the dimensions of the loaded area are large compared to the thickness of the compressible stratum e.g. under a fill. For other conditions, such as under a building load, a correction factor has to be applied which will be described later.

The consolidation settlement is estimated using the results of 1-D consolidation tests carried out in the laboratory on representative undisturbed samples. A field curve is established using Schmertmann's correction (see Bowles, 1982). From this curve, the ultimate consolidation settlement can be calculated by the following equation.

$$(\rho_c)_{final} = \sum_{\text{all layers}} \left\{ C_c \frac{H}{(1+e_o)} \cdot \log \frac{\sigma'_{vo} + \Delta\sigma'_v}{\sigma'_v} \right\}$$

where C_c = compression index
e_o = initial void ratio
σ'_{vo} = initial effective stress
$\Delta\sigma'_v$ = increase in effective stress
H = thickness of the cohesive layers.

The consolidation settlement at time t is given by

$$(\rho_c)_t = \frac{U}{100} (\rho_c)_{final}$$

where U is the degree of consolidation in percentage. For a given time t, if H is the length of the drainage path, the time factor T may be computed from

$$T = C_v \, t/H^2$$

and the corresponding value of U can be obtained from tables provided in any standard text book on Soil Mechanics and Foundation Engineering.

4.2.1 Constant Rate of Loading

The procedure explained earlier refers to the case of a loading situation which is instantaneous. However in practice it is far from instantaneous. Florin (See Leonards,1962) and Schiffman (1958) solved the time dependent loading problem analytically. However, Terzaghi suggested a simple procedure which is applicable to a constant rate of loading case (see Leonards,1962). As most loading sequences can be approximated to a constant rate, Terzaghi's solution is sufficient for any analysis in general.

4.3 Estimation of Secondary Compression

Secondary compression is generally considered to occur once the primary consolidation is over. In sands and low plasticity clays, the secondary compression is negligible. However, in highly plastic clays, organic clays and in peats, the secondary compression can be significant and therefore cannot be neglected in analysis.

The secondary settlement can be estimated from;

$$\rho_{secondary} = C_\alpha \, H_{field} \, \log \left\{ \frac{t}{(t_{100})_{field}} \right\}$$

where C_α is the coefficient of secondary compression given by:

$$C_\alpha = \frac{1}{H_{lab}} \left\{ \frac{\Delta H}{\Delta (\log t)} \right\}_{lab}$$

5.0 SETTLEMENT BEHAVIOUR OF BUILDINGS IN LOW LYING AREAS

How estimation could be made of settlements, immediate, preliminary consolidation and secondary consolidation, was discussed in the earlier sections of this paper. For these estimates to be realistic, conditions assumed in the model of soil behaviour and boundary conditions of the problem has to be appropriate. Since the actual behaviour in the field will depend on the conditions found at the site, a monitoring programme was carried out (Samaranayake et al, 1989) to observe the actual behaviour of buildings.

5.1 Monitoring of buildings

In the building monitoring programme under the research project a total of 135 buildings constructed at 34 different sites were monitored. However, monitoring had to be abandoned in 43 of these buildings at different stages due to practical difficulties. The buildings monitored were broadly categorised into load bearing walled and framed structures and further classified according to the foundation type as shown in Table 4. Typical cross sections of the foundation types used are shown in Fig. 3.

Table 4 : Classification according to Foundation Type

FOUNDATION TYPE	NUMBER
Masonry Strip	28
RC Strip with rubble foundation	8
Inverted - T	35
Vierendeel	12
Raft	1
Pad footing	51

5.2 Performance of Buildings in Low Lying Areas

Performance of the buildings monitored by NBRO are summarised in Table 5.

5.2.1 RC Framed Buildings

5.2.1.1 RC Strip Footings

Of the RC framed buildings monitored, only three were supported on RC strip footings. Two of them were two storeyed houses with floor area 54 m^2 and a L/B ratio of 1.3 which did not show any distress. The other was a warehouse with floor area 141 m x 35 m which showed severe distress including cracks,tilt and subsidence of the floor. The case history of performance of this building is discussed in 5.3.

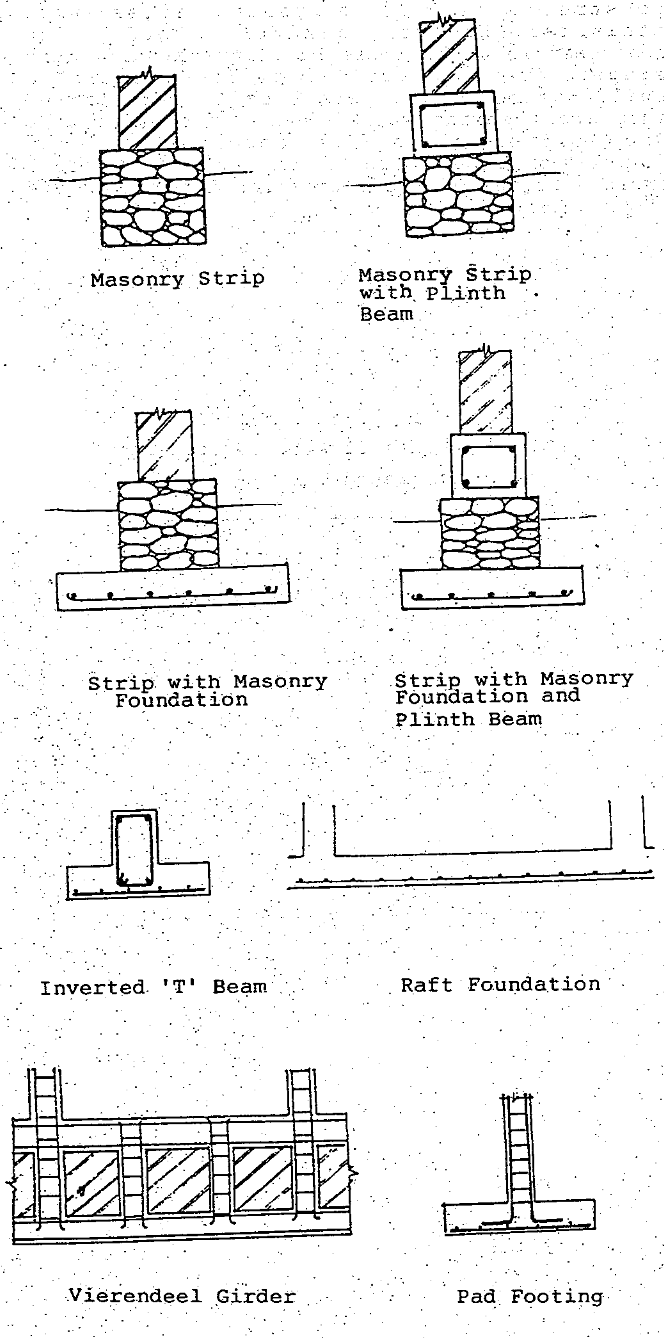


Fig. 3 Types of Foundations

5.2.1.2 Inverted - T Type Foundations

A total of 14 buildings with inverted T type foundations, most of which are located in Peliyagoda, Battaramulla and Bloemendhal zones were monitored.

Of thirteen two storeyed buildings constructed in a single site on a 2 m thick fill over a peat layer of thickness more than 5 m, 5 buildings showed only minor cracks during or after construction indicating that the foundation has so far performed fairly satisfactorily. In another four of the two storeyed buildings, which are presently constructed up to upper floor level showed no distress except for a few minor cracks in only one of them.

One building was a warehouse of double height, single storeyed structure, in which cracking was observed even before live load was imposed. Although the total length of the warehouse is 135 m, structural separations had been provided making it into three separate units. Maximum settlement of this structure exceeds 450 mm. Considerable distress noticed in the form of cracks on all wall panels indicate that the stiffened foundation used had not been very effective.

Table 5. Performance of Buildings

5(a) R/C Frame Structures

Type of foundation	Total No. of buildings	No. of storeyes		Buildings without cracks	Buildings with cracks		
		Single	Double		minor	moderate	severe
R/C strip	3	1	2	2	-	-	1
Inverted T	17	5	12	10	6	1	-
Vierendeel	9	6	3	4	3	2	-
Pad	22	15	7	5	1	15	1
TOTAL	51	27	24	21	10	18	2

5(b) Load Bearing Wall Structures

Type of foundation	Total No. of buildings	No. of storeyes		Buildings without cracks	Buildings with cracks		
		single	double		minor	moderate	severe
Masonry strip	26	26	-	18	6	2	-
R/C strip	3	3	-	-	2	1	-
Inverted T	10	10	-	8	2	-	-
Vierendeel	3	3	-	2	1	-	-
Raft	1	1	-	1	-	-	-
TOTAL	43	43	-	29	11	3	-

5.2.1.3 Vierendeel Girder Type Foundations

A total of nine buildings supported on vierendeel foundations, were monitored. Six of them are single storeyed and the remaining are two storeyed. Of the six single storeyed residential and office buildings three units showed minor cracks, and another was moderately cracked, while the remaining two did not indicate any distress. The moderately distressed building is discussed in section 5.3.

Of the two storeyed residential buildings, only one had shown any distress, in the form of minor cracks.

5.2.1.4 Pad Foundations

Of a total of 22 buildings, supported on pad footings 15 were single storeyed and seven were two storeyed.

Fourteen of the single storeyed buildings are double height structures where the walls (which are distressed with moderate cracks) are supported on RC strip footings, whereas the columns are built on pad footings. Therefore, in assessing performance of these structures, the effect of this pad and strip arrangement should also be considered.

The remaining single storeyed structure which is now at roof level has so far not shown any distress.

Of the seven two storeyed buildings only an office and laboratory building have shown minor cracks, while one residential building has shown moderate cracks. The remaining five residential buildings which have reached above the first floor slab level have not shown any distress so far.

5.2.2 Load Bearing Wall Type Structures

5.2.2.1 Masonry Strip Footings

Twenty six single storeyed buildings mostly residential, except two office buildings, supported on masonry strip foundations were monitored. Of these 17 units showed no distress, 6 showed minor distress and the others were moderately distressed. Although some of these buildings had settled more than 200 mm, the differential settlement had been small. Although there has been tilting in some of the buildings (rigid body rotation), this has been possible without other distress to the structure because most of these units are of small dimensions thereby increasing the stiffness of the structure.

5.2.2.2 RC Strip Footings

Of the three buildings on RC strip footings two had minor cracks and one was moderately distressed. Two buildings having cracks were office buildings still to be occupied, while the third building (having moderate distress) is a factory building. Although the settlement of the factory building was in the order of 150 mm, vibration caused by heavy machinery would also have contributed to the progress of cracks. No conclusions can be made because monitoring commenced after the factory started functioning.

5.2.2.3. Inverted - T Type Foundations

At a site in Bloemendhal ten buildings supported on inverted - T type foundations were monitored. All the buildings were found to have performed satisfactorily, with only two buildings which showed distress of minor nature.

5.2.2.4 Vierendeel Girder Type Foundations

At the same site in Bloemendhal, discussed above, vierendeel girder type foundations had been provided in three residential units. These had been located where the subsoils were found to be much weaker. The buildings had performed satisfactorily with only one unit showing distress of a very minor nature.

5.2.2.5 Raft Foundations

A rigid raft foundation had been provided for a single storeyed building in the Battaramulla zone, apparently because a portion of the land had been filled with waste material. As expected the rate of settlements of the building was high during the first five months of monitoring.

A maximum settlement of 200 mm was noticed when the monitoring had to be abandoned as desired by the client. However, no apparent distress was observed although negligible tilt had occurred.

5.3 CASE STUDIES

A few case studies that are considered important are discussed in this section. The information relevant to building layout structural and loading details, subsoil profiles, observed settlement and distress, predicted settlement, analysed data including relationships between average settlement with angular distortion and the L,M and N factors etc., are summarised in the annexures.

5.3.1. Office/Stores Buildings at Orugodawatta

Three buildings within a site at Orugodawatta were monitored commencing when the construction was at the roof level. Details of the site, subsoil conditions and information relevant to construction and settlement of a typical building are given in Annexure A. These single storeyed buildings vary from single height to double height and are supported on vierendeel foundations. All buildings have undergone distress of moderate nature as shown in Fig.6 of Annexure A. The predicted settlements using the methodology explained in this paper were conservative as seen from typical results in Fig.2 of Annexure A.

Although structural separations had been provided, the observations indicated that their performance had not been as anticipated. Further, because the L/B ratio and the overall dimensions had been high, stiffened foundation had not been fully effective.

5.3.2 Warehouse at Peliyagoda

Severe damages were observed in this warehouse and the rate of settlement had been very high. Distress was found as major cracks in the walls, tilts and settlement of the floor. The observations and information gathered indicate that the live load of 0.3 kg $/cm^2$ to 0.6 kg/cm^2 (which was cyclic) over the entire building area had contributed to the distress. This is because the subsoil profile varies considerably over the length of the building, with the thickness of underlying peat layer varying from 3 m to 10 m. This is shown in Annexure B.

5.3.3 Go-downs at Orugodawatta

At Orugodawatta two structures supported on pad and RC strip footings intended to be used as go-downs were monitored commencing when the structures were at roof level. The buildings have shown a few cracks as shown in Annexure C. The predicted settlements, taking into consideration the loading history and other available information, were higher than the observed especially in the initial stages.

5.3.4. Office Building at Peliyagoda

This is a two storeyed RC framed structure on pad footings and monitoring commenced from DPC level (see Annexure D). Predicted settlements were slightly higher than the observed values. The building was moderately distressed, and the maximum angular distortion has increased with average settlement.

5.3.5. Transit Accommodation at Hultsdorf

The scheme for the Transit Accommodation consisted of many single storeyed buildings with each building consisting of four units. The foundations were conventional rubble masonry type supported on top of a loose fill of 1.6 m to 2.0 m in thickness over a 1.5 m - 2.0 m thick organic clay/peat layer. Although loads were not high, considering the weak subsoils, and the very limited time available for construction, the structures were designed with provision for structural joints so that each unit would behave as an independent unit when settlements take place. The buildings had behaved satisfactorily with slight openings appearing along the structural joints as anticipated.

5.3.6 Residential Building at Rajagiriya

Two structurally separated adjoining buildings have been constructed as four housing units with two single storeyed and two double storeyed. The foundations are of vierendeel type used for the RC framed superstructure. Below a fill of 2 m, a soft organic clay layer of 1 m is found. Up to about 6 m depth the subsoils are of weak nature. Although high settlements had occurred the structure has performed satisfactorily, with only a minor crack.

5.3.7 Housing Scheme at Mattakkuliya

In this scheme consisting of 110 double storeyed housing units, 7 blocks had 8 units each and 1 block had 6 units. The buildings were load bearing masonry walled and resting on RC strip footings. Subsoils consisted of a lateritic fill (0.75 to 1.9 m) followed by a silty sand deposit mixed with organic matter. The very weak layers extended up to 5 m.

Severe cracks were found in some units while in some others low to moderate distress were observed. In some blocks, the rate of settlement was significant even after few years. The cause of distress was traced to high differential settlement due to weak, erratic nature of subsoils. It was also observed that the degree of rigidity in the buildings vary considerably between the longitudinal and the transverse directions. By dismantling several units at the middle of each block to increase the longitudinal rigidity of the remaining units, it was possible to arrest further distress in the buildings.

5.3.8 Single Storeyed Building at Kotte

Vierendeel girder system of foundation was adopted for this building on a 1 - 1.5 m lateritic fill on soft organic clay (Naganather, 1987) . As the expected settlements were high, the buildings were designed as five structure units. Each unit behaved separately, and although the settlements were very high, cracks appeared only at separations provided as intended. The satisfac ly performance of this building which is o. of the structures where vierendeel girder foundations were used in Sri Lanka has encouraged the use of stiffened foundations under similar situations.

5.4 Concluding Comments

One of the objectives of this research is to attempt developing of methods to predict the performance of buildings constructed in the reclaimed marshy areas. This was carried out by comparison of predicted settlement based on theoretical soil models, where available loading details and soil profile with assumptions used to simplify rigorous analysis, and with the field observations of settlements.

It was observed that predictions were not always in agreement with the observations. This may be due to one or many of the reasons given below:

1) Soil model used for analysis may not be appropriate especially for organic soils, their intricate behaviour and their high order of immediate, primary and secondary consolidation.

2) It is extremely difficult to obtain good estimates of required soil parameters due to many reasons including limitations of geotechnical investigations, non-homogeneous nature of soil profiles with pockets, seams, and lenses, difficulties and disturbances in sampling and testing which affect the results.

3) It is difficult to represent history and conditions of loading accurately in the field due to many reasons;

- history of fill cannot be easily traced.
- height of fill is variable due to intrusions into soft ground.
- history of building construction of existing buildings is difficult to obtain.

4) In the analysis described in this paper the soil-structure interaction was not taken into account. Due to the soil-structure interaction which inevitably occurs when a building is constructed over a yielding stratum, the exact pressure distribution under the foundations is far from uniform or linear. Although this phenomenon may not be significant for small settlements, its effect could be very high when settlements are large.

ACKNOWLEDGEMENTS

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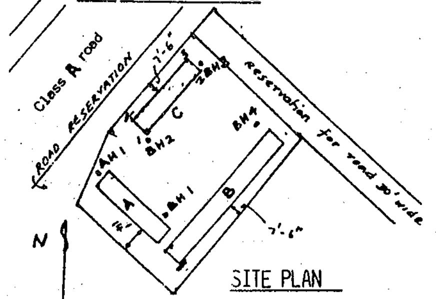
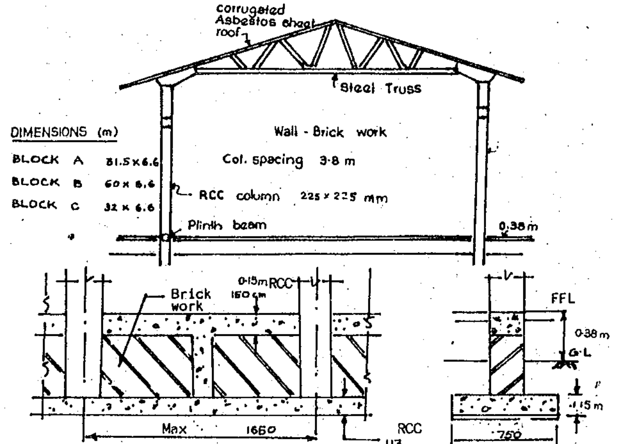
OFFICE/STORES BUILDINGS AT ORUGODAWATTE



SITE PLAN

Fig.1



Vierendeel Foundation

STRUCTURAL DETAILS

Fig.5

OBSERVED DISTRESS

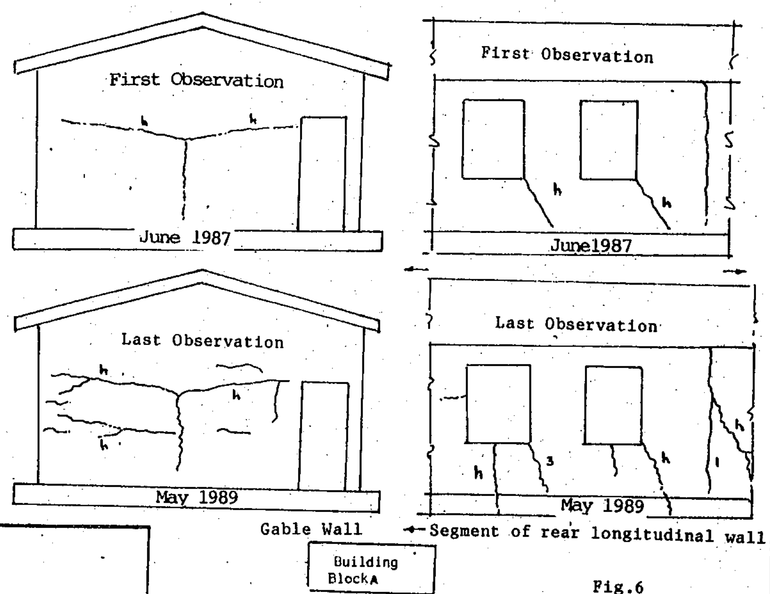
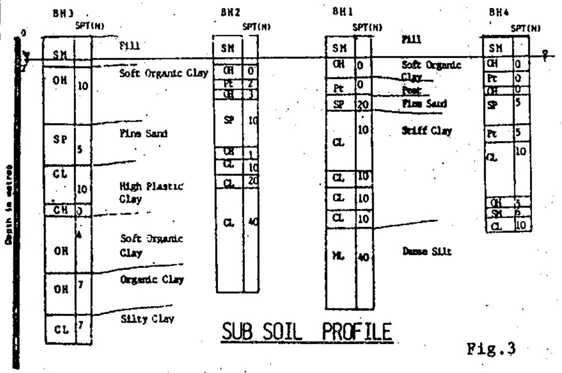


Fig.6



SUB SOIL PROFILE

Fig.3

PROPERTIES OF HIGHLY COMPRESSIBLE SOIL LAYERS

SOIL LAYERS	INITIAL VOID RATIO (e_o)	COMPRESSION INDEX (C_c)	BULK DENSITY (γ)	COEFFICIENT OF CONSOLIDATION (C_v)	COEFFICIENT OF SECOND. COMPRESSION C	MODULUS OF ELASTICITY (E)
			(g/cc)	(cm²/s)		(Kg/cm²)
Soft Organic Clay	2.375	0.99	1.35	10×10^{-4}	0.0014	35
Peat	2.21	1.172	1.22	0.5×10^{-4}	0.017	15

Fig.4

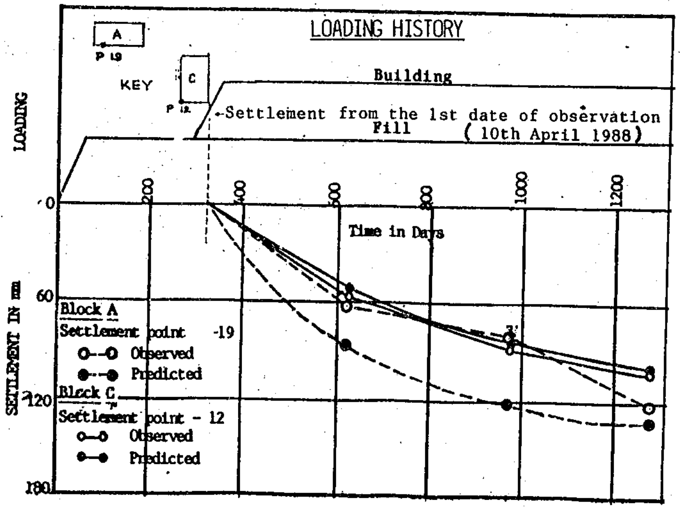
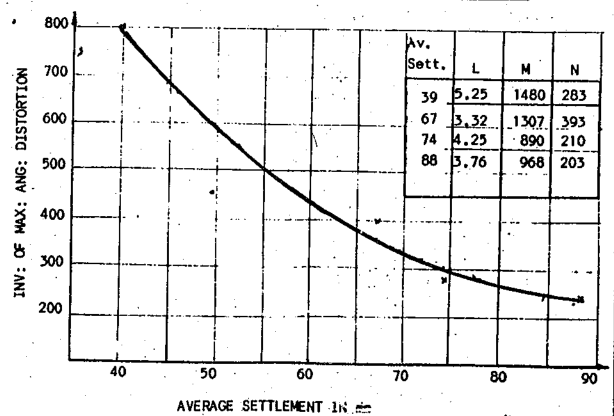


Fig.2

Comparison of calculated and observed Settlements from the 1st date of observation

RELATIONSHIP BETWEEN(1/Δ)& AVERAGE SETTLEMENT



Av. Sett.	L	M	N
39	5.25	1480	283
67	3.32	1307	393
74	4.25	890	210
88	3.76	968	203

Fig. 7

NOTES:

1. STRUCTURES: Site consists of three single storeyed RCC framed structures to be used as store rooms /office rooms (Fig.1-Site plan, Fig.5-Cross section). Structural joints are provided not to exceed the length to breadth ratio of 4.

2. FOUNDATION: Vierendeel Girder Type(fig.5)

3. SUB SOIL: Bore hole locations Fig. 1
   Bore logs Fig. 3
   Sub soil properties Fig. 4

4. PERFORMANCE: (a) Observed and calculated settlments for blocks A&C Fig.2
   (b) Cracks observed at different times in block A Fig.6
   (c) Relationship between inverse of Angular Distortion and Average settlement is shown in Fig.7

ANNEXURE A

WAREHOUSE AT PELIYAGODA

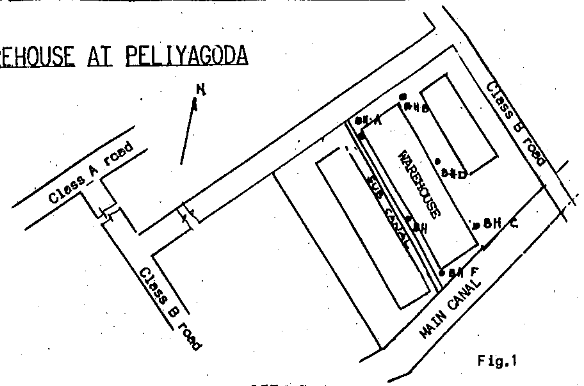


Fig.1

SITE PLAN

STRUCTURAL DETAILS

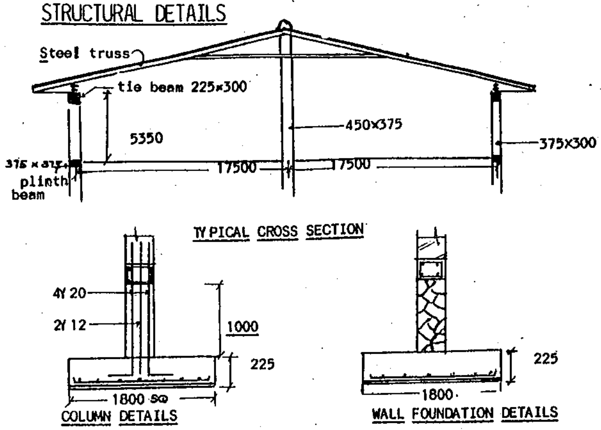


Fig.2

OBSERVED DISTRESS

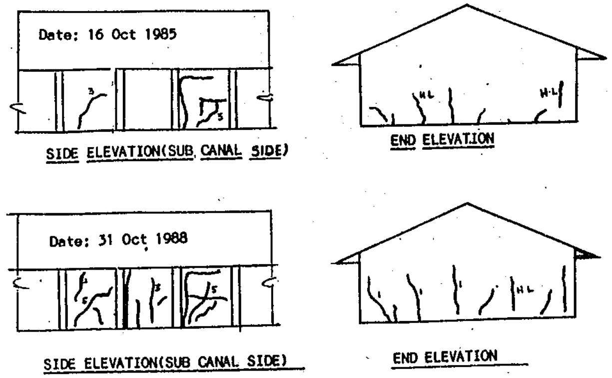
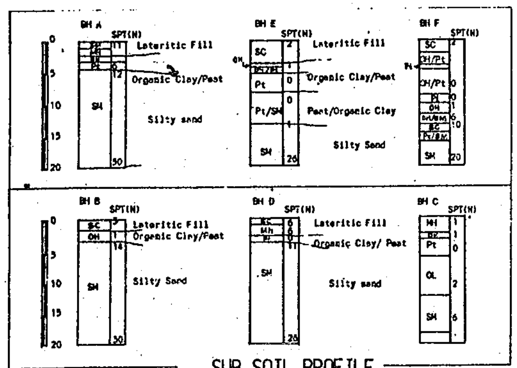


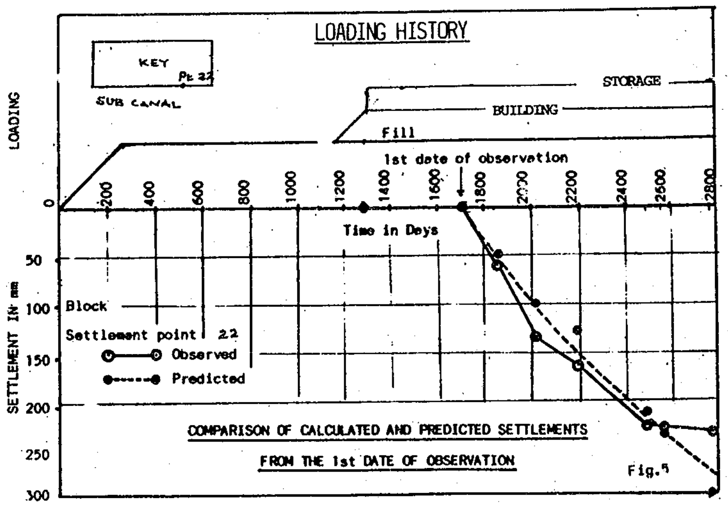
Fig.3



SUB SOIL PROFILE

Fig.4

LOADING HISTORY

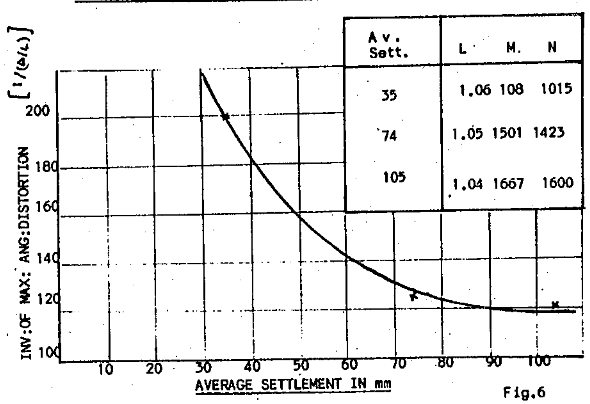


COMPARISON OF CALCULATED AND PREDICTED SETTLEMENTS

FROM THE 1st DATE OF OBSERVATION

Fig.5

RELATIONSHIP BETWEEN($\frac{1}{\Delta/L}$) & AVERAGE SETTLEMENT



Av. Sett.	L	M.	N
35	1.06	108	1015
74	1.05	1501	1423
105	1.04	1667	1600

Fig.6

PROPERTIES OF HIGHLY COMPRESSIBLE SOIL LAYERS

SOIL LAYERS	INITIAL VOID RATIO (e_o)	COMPRESSION INDEX (C_c)	BULK DENSITY (γ)	COEFFICIENT OF CONSOLIDATION (C_v)	COEFFICIENT OF SECOND. COMPRESSION C_α	MODULUS OF ELASTICITY (E)
			(g/cc)	(cm^2/s)		(kg/cm^2)
Soft organic clay	2.82	1.46	1.4	4.0×10^{-4}	0.007	30

Fig.7

NOTE:

1. STRUCTURES: The wrehouse is a single storeyed RCC frame structure with infill panels made of hollow blocks. The dimensions of the building are 141m x 35m. It consists of 4 bays which are used to store sugar.

2. FOUNDATION: Columns are provided with pad footing and the infill panels supported by rubble masonry strip with plinth beam Floors are separated from walls and consist of 6mx6m concrete slabs placed on a sand cushion Fig. 2

3. SUB SOIL : Bore hole locations Fig. 1
Bore hole logs Fig. 4
Sub soil properties Fig. 7

4. PERFORMANCE: This is one of the buildings where the observed distress is very high. Cracks are visible in almost every panel of the building and remedial measures have been taken to prevent collapsing

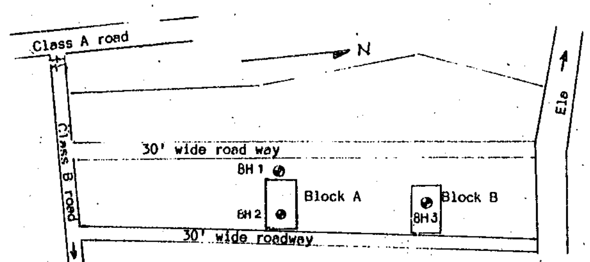
(a) Observed and calculated settlements are shown in fig. 5

(b) Cracks observed at different times are shown in Fig. 3

(c) Relationship between the inverse of Angular Distortion and Average settlement is shown in Fig. 6

ANNEXURE B

GODOWNS AT ORUGODAWATTE



SITE PLAN   Fig .1

STRUCTURAL DETAILS

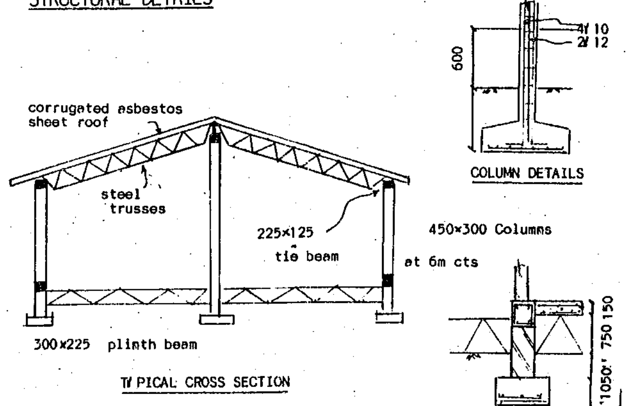


Fig.2

OBSERVED DISTRESS

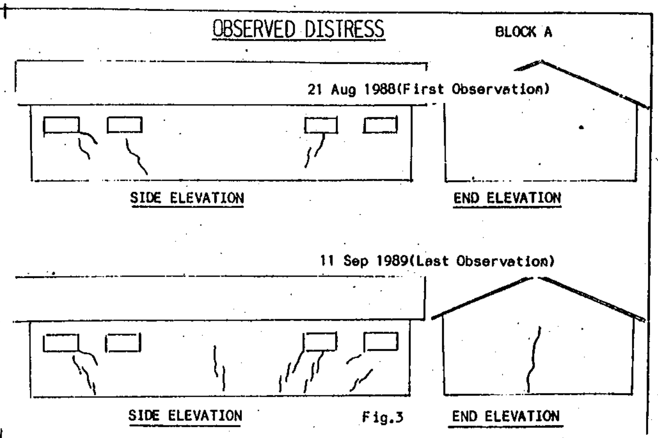
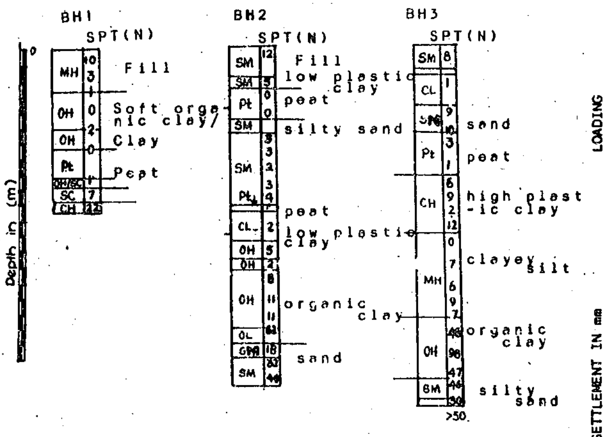
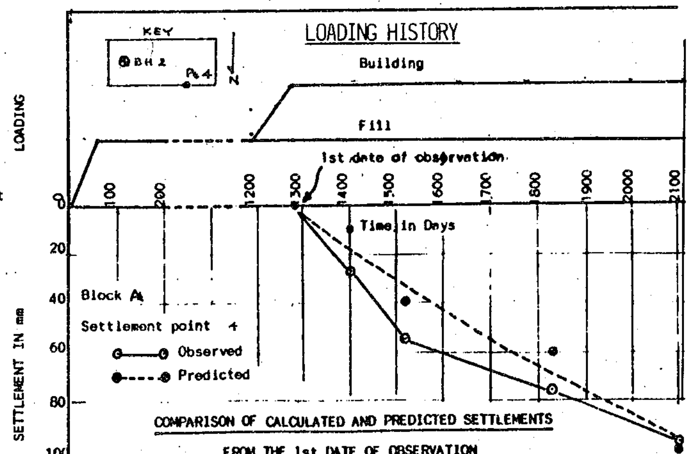


Fig.3



SUB SOIL PROFILE   Fig.4



COMPARISON OF CALCULATED AND PREDICTED SETTLEMENTS FROM THE 1st DATE OF OBSERVATION

Fig.5

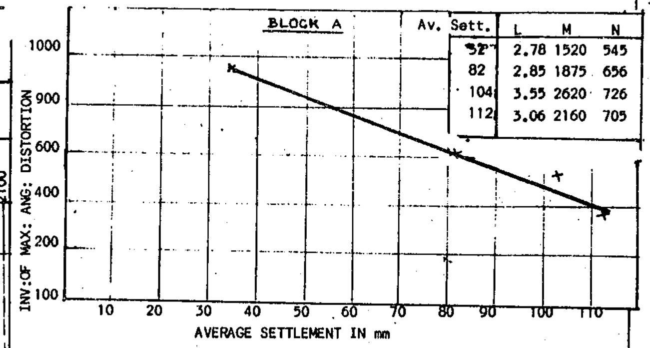


Fig.6

RELATIONSHIP BETWEEN(¹/δ/L) & AVERAGE SETTLEMENT

PROPERTIES OF HIGHLY COMPRESSIBLE SOIL LAYERS

SOIL LAYERS	INITIAL VOID RATIO (e_o)	COMPRESSION INDEX (C_c)	BULK DENSITY (γ)	COEFFICIENT OF CONSOLIDATION (C_v)	COEFFICIENT OF SECOND. COMPRESSION (C_α)	MODULUS OF ELASTICITY (E)
			(g/cc)	(cm²/s)		(kg/cm²)
Soft organic clay	1.8	0.7	1.4	3×10^{4}	0.0014	35
Peat	2.68	2.99	1.3	5.83×10^{-4}	0.009	15

Fig.7

NOTES:

1. Structures: Site consists of two identical single storeyed RCC framed structures to be used to store food grains. Fig 1.

Foundation: Columns are provided with individual pad footings while the walls are supported by a strip foundation. Fig. 2

Subsoil : Bore hole location Fig. 1
Bore hole logs Fig. 4
Compressibility properties Fig. 7

Performance: At the commencement of observation the buildings were constructed upto the roof level.

(a) Observed and calculated settlemenents are shown in Fig. 5

(b) Cracks observed at different times are shown in Fig. 3

(c) Relationship between the inverse of Angular Distortion and Average settlement is shown in Fig.6

ANNEXURE C

ANNEXURE D

OFFICE BUILDING AT PELIYAGODA

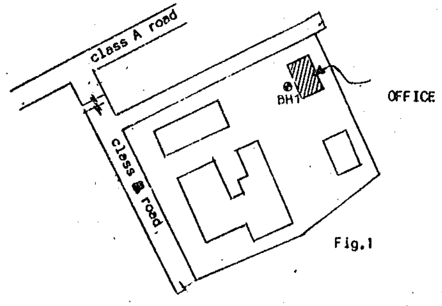


Fig.1

SITE PLAN

STRUCTURAL DETAILS

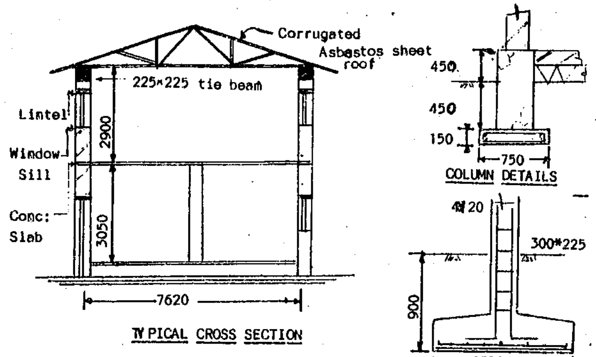


Fig.2

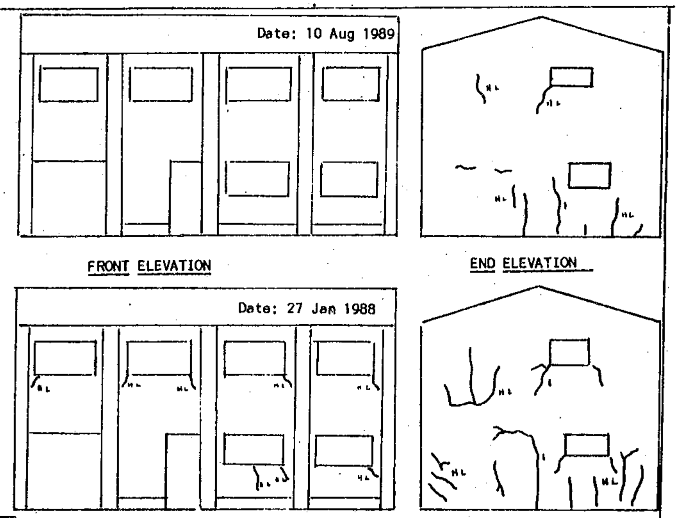


Fig.3

OBSERVED DISTRESS

SUB SOIL PROFILE

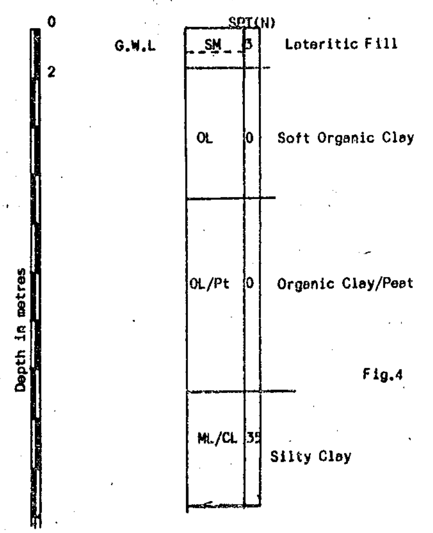


Fig.4

LOADING HISTORY

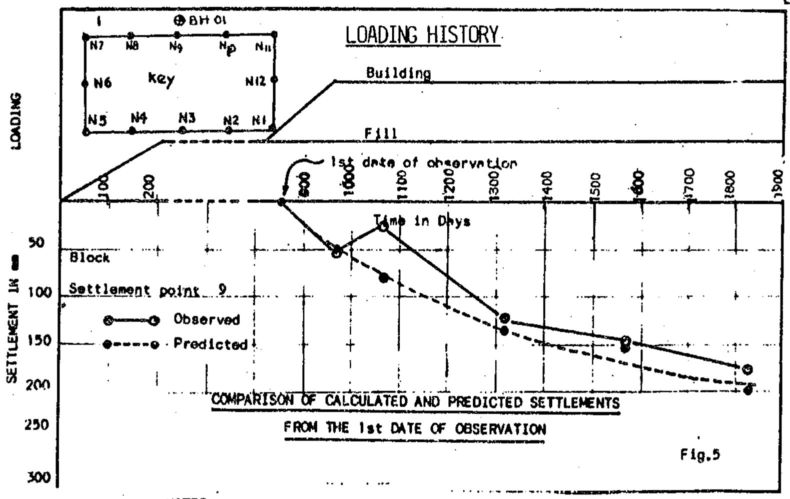


Fig.5

RELATIONSHIP BETWEEN(1/δ/L)& AVERAGE SETTLEMENT

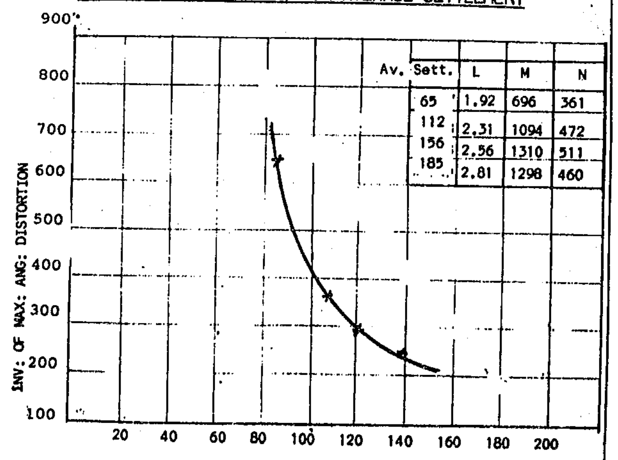


Fig.7

PROPERTIES OF HIGHLY COMPRESSIBLE SOIL LAYERS

SOIL LAYERS	INITIAL VOID RATIO (e_o)	COMPRESSION INDEX (C_c)	BULK DENSITY (γ)	COEFFICIENT OF CONSOLIDATION (C_v)	COEFFICIENT OF SECOND. COMPRESSION C_{α}	MODULUS OF ELASTICITY (E)
			(g/cc)	(cm^2/S)		(kg/cm^2)
Soft organic clay	4.48	2.59	1.4	52×10^{-4}	0.011	35
Peat	5.13	1.37	1.2	1.78×10^{-4}	0.015	15

Fig.6

NOTES:

1. STRUCTURE: The building is a RCC frame structure with 3 rows of columns spaced at not more than 4m at ground floor. All columns are provided with tie beams in two directions. The dimensions of the building are 12.5m x 8m.

2. FOUNDATION: Columns are provided with pad footings while the walls are supported by a strip footing. Fig. 2

3. SUB SOIL : Bore hole locations Fig. 1
   Bore hole logs Fig. 4
   Properties of compressible layers Fig. 6

4. PERFORMANCE: Monitoring of the office building started at D.P.C level.
   (a) Observed and calculated settlements are shown in Fig. 5
   (b) Cracks observed at different times are shown in Fig. 3
   (c) Relationship between the inverse of Angular Distortion and Average settlement is shown in Fig. 7

CONTD.

SOME RESULTS OF SOIL STRUCTURE INTERACTION STUDIES

B.L. TENNEKOON

In the routine method of structural design, the stresses and moments set up in a structure due to structural loading are calculated assuming unyielding foundations. However, most foundations (other than those founded on rock) do settle, and these settlements will cause a re-distribution of the stresses and moments within the structure. This is referred to as "soil structure interaction". The designer may choose to ignore this and design the structure on the assumption of unyielding supports, but the interaction will nevertheless take place. While this may be of no consequence for small settlements, its effect may be more than envisaged when the settlements are large.

A study of the literature shows that there are two broad objectives in carrying out soil structure interaction studies. They are

(i) the need to estimate the form and magnitude of the differential settlement which information is then used to assess the likelihood of damage and;

(ii) the calculation of the distribution of forces and stresses within the structure

This paper covers both aspects of these studies.

PART I

ESTIMATION OF FORM AND MAGNITUDE OF DIFFERENTIAL SETTLEMENTS AND ASSESSMENT OF LIKELIHOOD OF DAMAGE

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 50mm in sands and 75mm 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 constructed safely provided that the differential settlements are kept within allowable limits by providing adequate stiffness to the structure.

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.

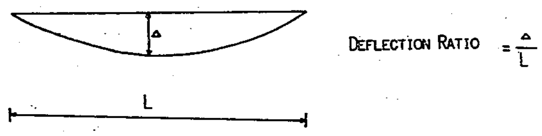


Fig 1a - Definition of Deflection Ratio for Load Bearing Wall Structures

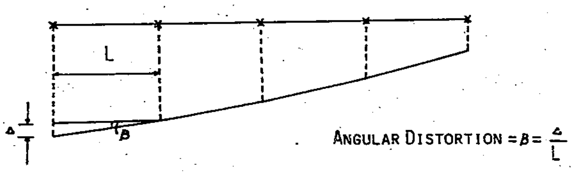


Fig 1b - Definition of Angular Distortion for Framed Structures

2.1 Limiting Deformation Critterion for Load Bearing Wall Structures

BURLAND AND WROTH (1975) have from theoretical considerations shown that the limiting deflection ratio for load bearing wall structures 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 been 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 leveling 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

Nawagampura Block D

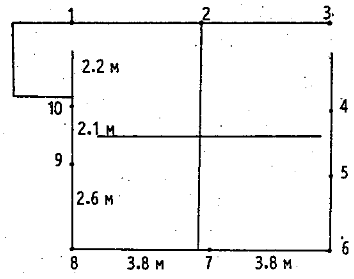


Fig 2a - Plan view of Building

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 Δ = 4 mm L = 6.9 m

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

Fig 2b - Computation of Deflection Ratio

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 commence at a deflection ratio of (1/2750).

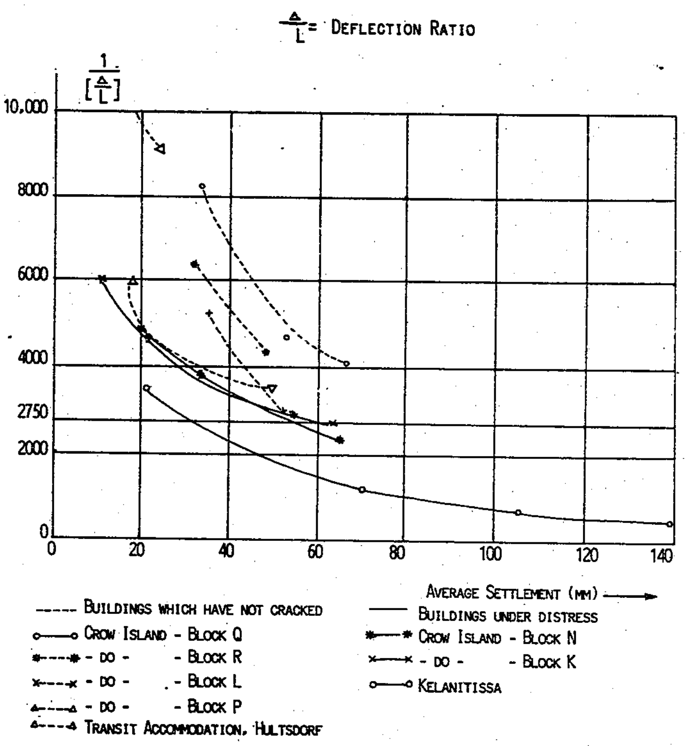


FIG 3a - RELATIONSHIP BETWEEN DEFLECTION RATIO & AVERAGE SETTLEMENT FOR LOAD BEARING WALL STRUCTURES

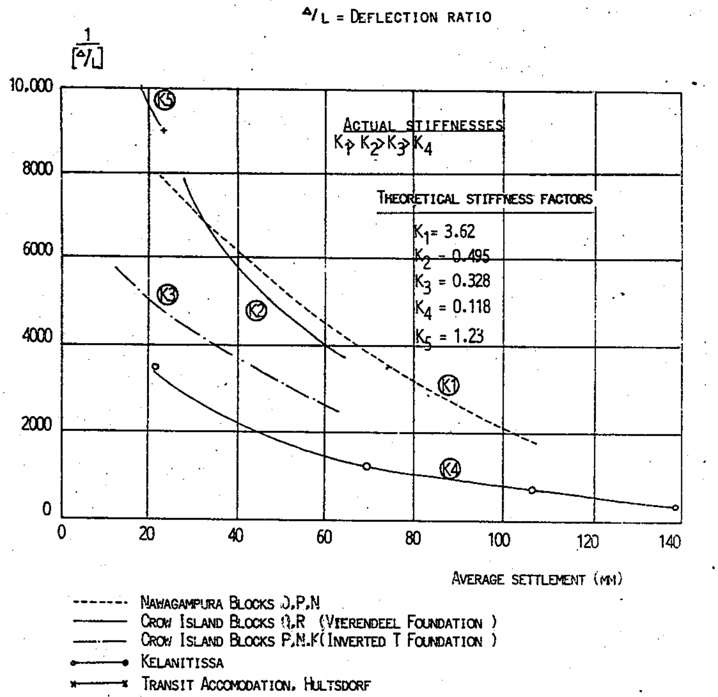


FIG. 3c - RELATIONSHIP BETWEEN DEFLECTION RATIO & AVERAGE SETTLEMENT FOR LOAD BEARING WALL STRUCTURES

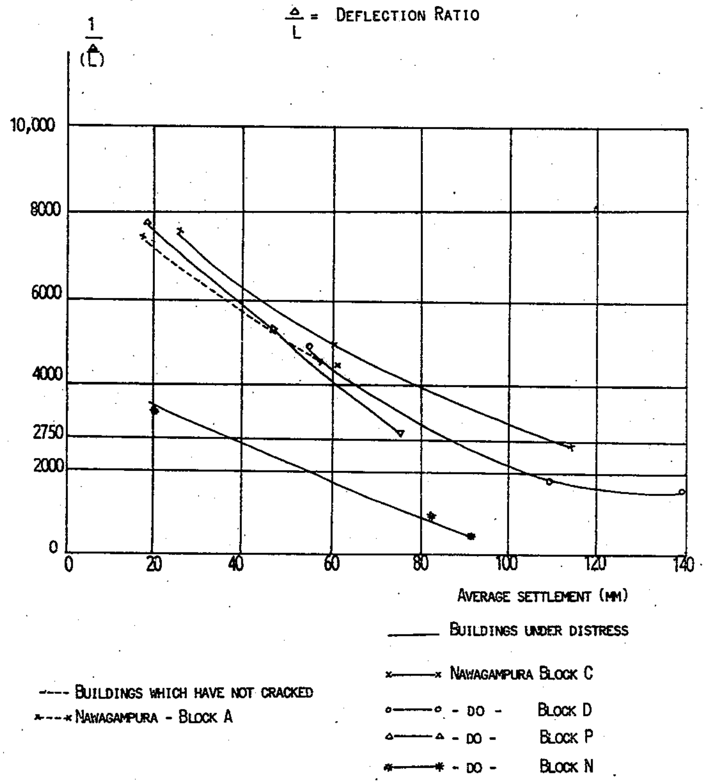


FIG. 3b RELATIONSHIP BETWEEN DEFLECTION RATIO & AVERAGE SETTLEMENT FOR LOAD BEARING WALL STRUCTURES

For purposes of comparison, 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 a typical example, the plan of the IDB Building at Peliyagoda is shown in Fig. 4a. Settlement markers have been placed at the 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.

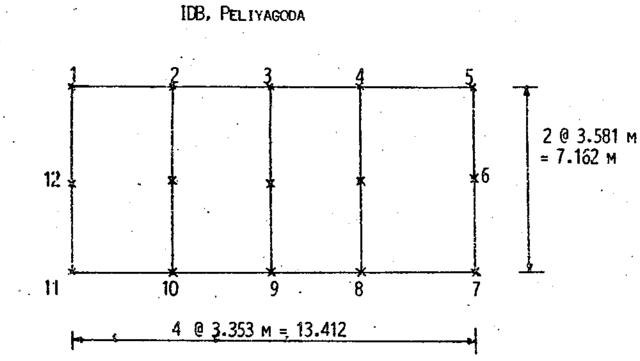


FIG 4 a - PLAN VIEW OF BUILDING

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

$\Delta_{MAX} = 13$ MM

$$\beta_{MAX} \quad \frac{13}{3.353 \times 1000} = \frac{1}{258}$$

FIG 4b - COMPUTATION OF MAXIMUM ANGULAR DISTORTION FOR FRAMED STRUCTURES

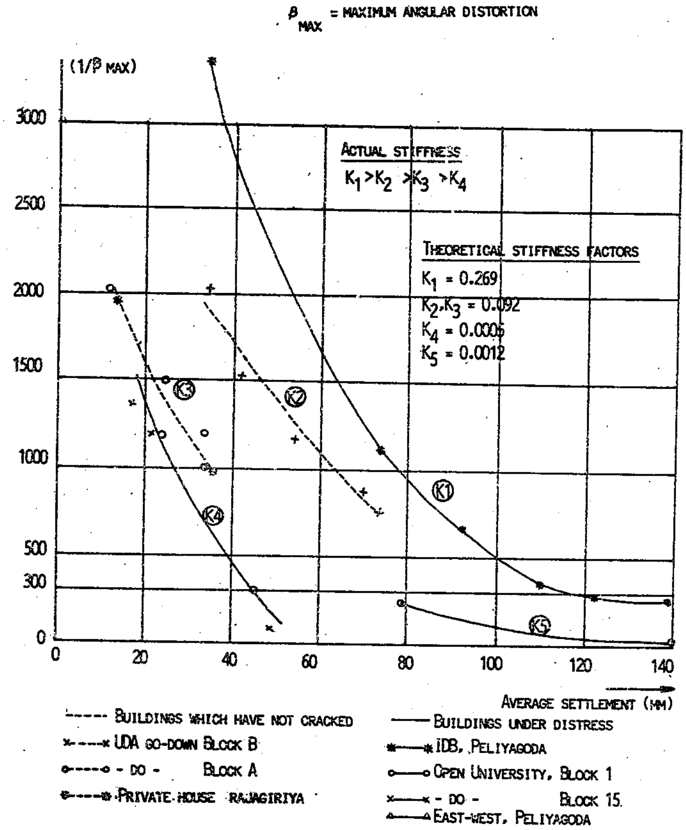


FIG.5 - RELATIONSHIP BETWEEN ANGULAR DISTORTION & AVERAGE SETTLEMENT FOR FRAMED STRUCTURES

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 WROTH (1975) all report a limiting angular distortion of 1/300 for framed structures.

These results also show that the framed structures can withstand larger differential settlements than load bearing wall 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 =

$$\frac{E_w \times I_w \times b^2}{2H^2}$$

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

$$= \sum E_b I_b \left[1 + \left(\frac{K_l + K_u}{K_b + K_l + K_u} \right) \frac{b^2}{l^2} \right]$$

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

K_l, K_u, K_b are the average stiffnesses 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;
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 stiffnesses of the structures as theoretically determined are shown in Tables 1a and 1b. It is seen from Table 1a 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 make 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, stiffnesses 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 a vierendeel type foundation system. As an example of this, of the structures analysed, in the case of the house at Rajagiriya 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 structure. 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 differential settlement.

Project	Stiffness of Structure			Stiffness of Soil				Relative Stiffness Factor (K)
	(EI) walls ($kN.m^2$)	(EI) frame ($kN.m^2$)	(EI) total ($kN.m^2$)	E_s (kN/m^2)	a (m)	b (m)	(EI) Soil ($kN.m^2$)	
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. Private House, Rajagiriya	8.7×10^6	0.1×10^6	8.8×10^6	10000 20000	7.54	9.98	75×10^6 150×10^6	0.118 0.059
4. East-West Peliyagoda	7.8×10^6	0.1×10^6	7.9×10^6	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	8.258x10^6	2000	2.6	7.6	2.283x10^6	3.62
2. Crow Island, (Vierendeel)	91.77x10^6	5000	3.81	21.34	185.1x10^6	0.495
3. Crow Island, (Inverted T)	60.76x10^6	5000	3.81	21.34	185.1x10^6	0.328
4. Devco Showa	16.9x10^6	5000	9.0	27.0	885.7x10^6	0.019
5. Transit Accommodation	10.8x10^6	2000	6.9	9.0	8.748x10^6	1.23
6. Kelanitissa, Block A	36.57x10^6	2000	9.9	25	309.4x10^6	0.118

Table 1b - Stiffness determination for Load Bearing Wall Structures

4.0 RELATIVE STIFFNESS OF STRUCTURE

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 parameter K which can be used as a measure to evaluate differential settlements.

MEYERHOF (1953) has derived theoretically a relationship between $(\frac{\Delta max}{\rho})$ 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 $(\frac{\Delta max}{\rho}) \simeq 0.08$. ($\Delta$ max is the maximum differential settlement, and ρ is the average settlement).

For the stiffness of the soil, MEYERHOF (1953) has obtained stiffness of soil = E_sx b^3x 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 being adopted by Indian Standards Institution (ANON, 1972).

This 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 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.

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 Static Cone Test in which MEYERHOF (1965) has recommended the use of the empirical relationship

$$E_s = 1.9q_c$$

where q_c is the Static Cone resistance.

In the absence of Static Cone resistance results, the recommendations of the Indian Standard Institution - ANON (1972) - have been used. According to this recommendation, for loose sand;

$$E_s = 200 \text{ to } 500 \text{ kg/cm}^2$$

i.e. E_s = 20,000 to 50,000 kN/m^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, Orugodawatta. These tests gave

$$c_u \doteq 0.35 \text{ kgf/cm}^2$$

Taking E_s = 50 x cohesion

E_s for the peaty soil works out as 1700 kN/m^2.

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, Orugodawatta.

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 secant 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/m^2.

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	7,000 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)

The stiffnesses 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 stiffnesses 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 Orugodawatta 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 - 152mm) whereas the UDA Go-downs had been designed for a bearing pressure of 160 kN/m^2 and had undergone a settlement of (60 -87mm).

This is despite the fact that the thickness of the two peat layers are 1.98m for the IDB Building site, and 5.90m 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 has 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, Orugodawatta 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 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. 3a and 3b, 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 = 25mm) has resulted in extremely small differential settlements (Δ= 2mm), 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. Two possible reasons are,

(i) the Nawagampura houses were found to be poor in their workmanship, and hence the actual stiffness of the structures would have been reduced; and/or

(ii) the increase in Relative Stiffness Factor beyond 0.5 has 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).

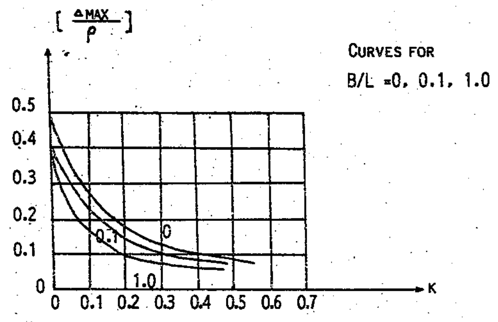


FIG. 6 EFFECT OF RELATIVE STIFFNESS FACTOR ON DIFFERENT SETTLEMENT OF UNIFORMLY LOADED RECTANGULAR FOOTING- (MEYERHOF, 1953)

5.0 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 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.

6.0 ACKNOWLEDGMENTS

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APPENDIX 1

List of structures which have been studied

Framed Structures

1. I.D.B., Peliyagoda
2. U.D.A. Go-downs, Orugodawatta - Block A
3. U.D.A. Go-downs, Orugodawatta - Block B
4. Private house, Rajagiriya
5. Open University, Nawala - Block 1
6. Open University, Nawala - 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 2.90 m has been placed above it.

The E values for the lateritic fill and the peat have been estimated as 50,000 kN/m^2 and 1700 kN/m^2 respectively.

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

$$\text{i.e. } 0 < (z/b) < 1.64, \quad \frac{1}{E} = \frac{1}{50,000} = 0.2\times10^{-4}$$

Similarly in the range $2.0 < z < 7.90$

$$\text{i.e. } 1.64 < (z/b) < 6.48, \quad \frac{1}{E} = \frac{1}{1700} = 5.88\times10^{-4}$$

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

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

This gives $\bar{E} = 5130$ kN/m^2.

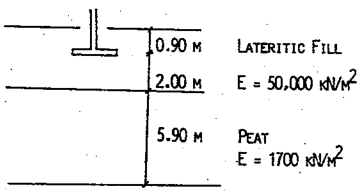

FIG. 7a - GROUND CONDITIONS AT UDA GO-DOWNS, ORUGODAWATTA

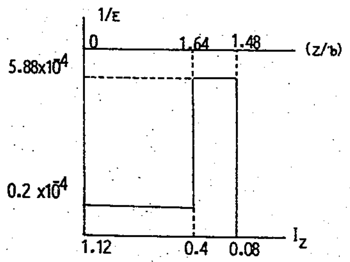

FIG. 7b - RELATIONSHIP BETWEEN 1/E & I_Z USING RESULTS OF FRASER & WARDLE (1976)

PART II

CALCULATION OF THE DISTRIBUTION OF FORCES AND STRESSES WITHIN THE STRUCTURE

1.0 INTRODUCTION

Mathematical modelling is an essential part of the development of analytical methods for structural analysis and design. This part of the paper deals with a critical study of the computer methods which have been developed (based on these models) for the calculation of the forces and stresses within the structure.

2.0 IDEALIZATION AND REALITY

With the advent of high speed, large storage computers the stage has now been reached when it would soon be possible to provide theoretical solutions to most problems in Structural Mechanics when given the geometry, the material properties, and the loading. However, it must be kept in mind that in arriving at these solutions idealizations have been made with respect to both the soil and the structure. There are idealizations concerning soil geometry, soil properties, resultant foundation loads, structural geometry, structural loading, structural properties etc. The fact remains that the uncertainties in both the soil and the structure (vis a vis these idealizations) are so great, that even with the possession of unlimited computer power it is unlikely that the precision in the prediction of behaviour will improve significantly. Therefore, for the purpose of this study, the methods of analysis have been divided into two categories - 'Detailed Analysis' and 'Simple Analysis'. 'Detailed Analysis' consists of the use of methods which involves the use of complex mathematical models; while the 'Simple Analysis' involves less sophisticated modelling but it is supplemented by the results of observed behaviour of structures.

3.0 DETAILED ANALYSIS

The detailed analysis involves the mathematical modelling of the three basic components which are the structure, the foundation, and the soil. They have to be separate but interdependent models.

3.1 The Structure Model

For soil structure interaction studies, it is necessary to evaluate the overall stiffness of the structure as a whole rather than the stiffness of individual elements as is usually done. The typical structure is an extremely complex assembly of elements whose stiffness varies with time especially during the construction process. Therefore, approximations have to be used when modelling the structure. For example, it is necessary to know what allowance should be made for the walls, partitions and floors in framed structures.

An approximate method for modelling the stiffness of framed structures was given by MEYERHOF (1953); and 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 a study of the settlements of buildings in the low lying areas of Colombo, TENNEKOON, SIVAKUGAN AND LAKSHMAN (1988) found that the theoretical stiffnesses as determined using the method of Meyerhof corresponded reasonably well with the actual stiffnesses of the buildings as observed from the settlement patterns.

3.2 The Foundation Model

In most analyses, the foundation is considered as a plate or beam in contact with the ground and its stiffness is computed accordingly. However, it is clear that this is erroneous because the bending of the foundation plate or beam will also be governed by the contribution to stiffness provided by the superstructure. 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 load bearing wall construction founded on a 0.76 m thick raft. Even after using very sophisticated detailed analysis, satisfactory agreement between measured and computed differential settlements was possible only after the structure was converted to an equivalent raft of thickness 4.6 m.

3.3 The Soil Model

In the usual method of foundation design, the stresses in the foundation are computed assuming a linear distribution of contact pressure. But it is well known that the contact pressure distribution is a complicated function of the interaction between the foundation (inclusive of structure) and the soil.

A study of the literature shows that there are three principal ways of modelling the soil. They are as

(i) a set of linear unconnected springs;
(ii) a half space continuum; and
(iii) a layered continuum.

All these models make use of soil properties which are averaged over the site, and these variations could make the modelling meaningless unless used with proper engineering judgement. Several computer programmes are available for analysing the stresses in the foundation using the different assumptions.

4.0 COMPUTER PROGRAMMES FOR USE IN SOIL STRUCTURE INTERACTION STUDIES

4.1 Soil Model as a Set of Linear Unconnected Springs

In soil structure interaction studies, the simplest method of solution is to model the soil as a set of linear unconnected springs. This is commonly referred to as the Winkler method, and the contact pressure at any point in the foundation is assumed to be proportional to the settlement at that point.

Computer programmes have been developed by BOWLES (1974) for

(i) combined footings using finite difference formulation;
(ii) combined footings using finite element formulation;
(iii) raft foundations using finite difference formulation;
(iv) raft foundations using finite element formulation.

All these programmes have been compiled by the Author on an IBM PC. In the programme for the raft foundation using finite element formulation, the method of matrix inversion for the solution of the equation

$$[X] = [K]^{-1}\ [P] \qquad \text{Equation (1)}$$

was modified. The use of the Choleski method of matrix inversion resulted in considerable saving of computer time (In the above equation, [X] is the displacement matrix, [K] is the stiffness matrix, and [P] is the load matrix.).

When the Winkler method of solution is used, the deflected form of the foundation is first determined by the solution of Equation (1). The bending moments and shear forces in the foundation beam or plate are then computed from the equations

$$M = EI\,\frac{d^2y}{dx^2} \quad \text{and} \quad S = EI\,\frac{d^3y}{dx^3}$$

In the formulation given by Bowles, the stiffness of the super-structure has not been considered for the determination of the deflected form; i.e. the stiffness matrix [K] in Equation (1) has been formed by considering only the stiffness of the foundation. The Author has dealt with this problem by evaluating an equivalent thickness of foundation after taking into consideration the stiffness of the structure and the foundation. This has been done by using a suitable approximation for the structure stiffness; e.g. using the approximation of MEYERHOF (1953).

Another drawback of this method (which also applies to other elastic methods of analysis) is that in the soil model no account is taken of 'failure' of the soil. At regions in the soil which are highly stressed and which have reached the failure criterion there will be a re-distribution of stresses with the shedding of load from the highly stressed regions to the less stressed regions. Therefore, the Winkler method is best suited for the situation when the foundation pressures are much less than the ultimate bearing capacity of the soil. TENNEKOON (1989) has shown that this is usually the case of buildings in marshy areas where foundations are more likely to fail by excessive settlement rather than by shear failure of the soil.

4.2 Soil Model as a Half Space Continuum

When modelling the soil as a half space continuum, usually elastic theory is used, and mathematical models are formulated to relate the forces and moments acting at the common interface between the foundation and soil to the corresponding displacements and rotations. As the computer programmes based on this model are not available with the Author, these will be referred to only briefly for completeness sake.

When considering the many methods that have been used to solve the equations governing the behaviour at the interface, two basic techniques can be identified. These are called the 'Iterative methods' and the 'Non-iterative methods'.

4.2.1 Iterative methods

In these methods, the common interface between the soil and the structure is broken up into a series of elements using carefully selected nodes. Then using standard methods of analysis in Structural Mechanics a relationship is obtained between the net applied force and the displacements at the interface nodes. Similarly, using well known Soil Mechanics procedures a relationship is obtained for the surface settlements at these nodes in terms of the yet unknown ground reactions. These two sets of relations are compared and iterative methods are used to establish common displacement profiles and ground reactions at the interface; e.g. ZBIROHOWSKI-KOSCIA AND GUNASEKARA (1970).

4.2.2 Non-iterative methods

With the development in recent years of structural analysis techniques using finite elements and the availability of powerful computers more direct methods of solution have been developed in which the common displacement profile and ground reaction are determined in a single discrete step. In general, both the structure and the ground must be considered three-dimensionally; for example, KING AND CHANDRASEKARAN (1975); MAJID AND CUNNELL (1976). However, such an analysis requires considerable computer power, and often a simplification is made for the structure where it is considered as a series of independent plane frames.

An alternative method of analysis developed by FRASER AND WARDLE (1976), where the structure is modelled using finite elements but the soil is modelled as a continuum will be discussed in Section 4.3.

4.3 Soil Model as a Layered Continuum

FRASER AND WARDLE (1976) developed an integral transform method to evaluate the stresses and displacements of a loaded area in an elastic medium. In this very powerful method, double Fourier transforms are used to reduce the partial differential equations of equilibrium to a set of ordinary differential equations which are, therefore, easy to solve. Advantages of this method are that it allows

(i) the analysis of a multi-layered soil system with different elastic properties in each layer. The layers can be of variable thickness.

(ii) different elastic systems to be considered; e.g. the soil may be considered as isotropic (as is usually done), or it may be cross-anisotropic where the elastic properties in the vertical direction are different from those in the horizontal direction (a common feature in many sedimentary deposits).

However, the soil layers have to be horizontal, and each layer has to be of uniform thickness and of infinite lateral extent. The program FOCALS (Foundation On Cross Anisotropic Layered System) which is commercially available is based on this method. (The source code and the executable programme are available with the Author.).

In the analysis, a set of equations of the form

$$[K]\ [X] = [P]$$

are set up for the different elements in the structure and the soil . (The notations are the same as that given earlier in Equation (1).).

The stiffness of the structural elements (which could consist of one or more of the following) are evaluated using the well known SAP programmes developed at the University of California, Berkley (WILSON (1971); BATHE, WILSON AND PETERSON (1973)) :

(i) three dimensional truss;
(ii) three dimensional beam;
(iii) plane stress and plane strain;
(iv) two dimensional axi-symmetric solid;
(v) three dimensional solid;
(vi) plate and shell;
(vii) boundary (ie. spring elements);
(viii) thick shell elements.

In the case of the soil, the entire soil mass is considered as a single element for which the stiffness matrix is evaluated using the integral transform method.

5.0 SIMPLE ANALYSIS

Detailed analyses whilst being very sophisticated and also expensive may not always give the correct answers to an engineering problem. It is worth quoting from a paper by PECK (1972) on 'Field Instrumentation and Measurements in Applied Soil Mechanics'.

"Instrumentation has become a catchword; field observations are recommended profusely by professionals in applied Soil Mechanics, and are even looked on with favour by many clients who ultimately pay the bills. But do the sponsor's get their money's worth? Sometimes, unmistakably yes. All too often no.

....The measurements may be reliable, the devices may function perfectly, and an ample number of observations may be carried out. The information may even be exquisitely presented, but all to no useful purpose because the data lack some vital quantity needed to understand the significant happenings on the job.

....The foregoing remarks suggest that measurements, even if dignified by calling them the results of instrumentation, do not in themselves make for improved understanding or better practice".

Unless judgement is carefully used, what field instrumentation and measurements are to applied Soil Mechanics, computer methods of analysis may become to structural design.

5.1 Some Simple Analytical Methods of Foundation Design Based on the Results of Observed Behaviour of Structures

At present, the design of foundations is based on limiting the total settlement (ρ), although it is the differential settlement (Δ) which causes distress to a building. The differential settlements can be controlled by providing adequate stiffness to a structure. The non-dimensional parameter called the Relative Stiffness Factor (K), which is the ratio of the stiffness of the structure to the stiffness of the soil, is well suited for soil structure interaction studies.

Theoretical relationships have been established between the differential 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 superstructure to overall structure stiffness has not been taken into account.

TENNEKOON, SIVAKUGAN AND LAKSHMAN (1988) report on a study carried out at the National Building Research Organisation where the settlement patterns of a large number of buildings in and around Colombo have been studied. They conclude that the limiting deformation criteria for the onset of cracking in plaster are :

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

From these results, and the structure stiffness computed according to the method of MEYERHOF (1953), they obtained the relationship between K and at failure. TENNEKOON, SENANAYAKE AND AMERATUNGA (1989) have shown that these results can be made use of to evolve a design method which can assess the likelihood of damage.

TENNEKOON AND RAVISKANTHAN (1988) have proposed an alternative method for computing the bending moments and stresses in the foundation. Rather than design the foundation for carrying the net loads, it is proposed that they be designed for the limiting state when cracking will occur; i.e. the foundation design is based on a limiting deformation criterion. Using this method, the foundations of load bearing walls can be designed using simple hand calculations, while in the case of framed structures the computer programme MICROFEAP may be used with the limiting foundation settlements introduced as part of the input data.

6.0 CONCLUSIONS

Mathematical modelling is an essential part in the development of analytical methods for structural analysis and design. It is known that with the rapid development of powerful computers, it is now possible to find theoretical solutions to virtually all problems in Structural Mechanics. However, because of the idealizations (and approximations) made in the mathematical model, the precision in the prediction of behaviour will not necessarily improve significantly. Detailed analysis based on modelling the structure and the soil in a variety of ways is discussed. Reference is made to some of the computer programmes which are available and these are analysed. In view of the complexity of the problem of soil structure interaction, it is shown that simple methods of analysis also have an important part to play in foundation design. Finally reference is made to some of the Author's work where simple methods of analysis have been used to assess the likelihood of damage, and to determine the foundation stresses as a result of settlements.

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SUGGESTED PRACTICE FOR THE CONSTRUCTION ACTIVITIES IN LOW LYING AREAS

A. THURAIRAJAH

INTRODUCTION

A large extent of land and floor space have become necessary in Colombo city and its vicinity as a result of the increase in population and the rapid development programmes that were launched specially during the past decade. But, with practically all the buildable highlands being already developed, the planners' attention has been focussed on the development of low lying marshy areas, and accordingly approximately 1600 ha of low lying lands in Colombo region had been earmarked to be developed in order to cater for the land demand.

As it had been already iterated in this seminar, a significant feature of these marshy areas is the presence of organic subsoils, predominately peat, distributed over considerable depth and breadth. A significant fraction being vegetable matter in various stages of decomposition, these soils are not regarded as suitable foundation material. Technological and economical conditions in countries like Sri Lanka would not permit utilization of these lands, for example, by complete replacement of the weak subsoils with quality materials or by improving the subsoils by solidification etc., as many an engineer in a developed country would be willing to do. Therefore, avoidance of these lands for building purposes by the engineers in the past, appears to be reasonable and quite understandable when one reconnoitres the low lying areas.

It is not surprising to locate many a building built on low lying areas with some form of distress, and at least a few showing major distress that may even endanger the safety. On the other hand one may surprisingly find well-standing buildings among such distressed buildings. Reasons for such performance need to be explored. Though, minor distress may be attended with simple repairs at low cost, often redemption or renovation of a severely distressed structure could be even more costly than reconstruction, specially when underpinning and other complicated foundation techniques are involved.

Situations could be encountered where lands with sound subsoil conditions are under-utilized for low rise, low cost buildings, while on the contrary, large superstructures which transmit heavy loads to the ground are constructed on lands with extremely poor subsoils after making enormous expenditure on substructure which may involve deep foundations and ground improvement.

Therefore the Geotechnical Engineer has a great responsibility in guiding the City Planners and Developers on optimum land use by imparting the knowledge on the suitability of lands, especially the reclaimed low lying marshy areas, for development. To meet this challenge, comprehensive information with regards to the nature, distribution and engineering properties of the subsoils is required by the engineer to assess the engineering response of such ground to the development activities envisaged. However, such comprehensive information was not readily available in the past although records of a limited number of isolated investigations carried out on ad-hoc basis were available at a few agencies. A pioneering effort has been made by the Central Soils Testing Laboratory of the Urban Development Authority in 1982 to document subsoil information on low lying areas (Ray and others, 1982). However, the real fruits of this attempt became evident only after 1984, when several research projects were embarked upon by the National Building Research Organisation (NBRO) with the assistance of the United Nations Development Programme (UNDP). These research projects are;

1) Geotechnical Mapping of Low Lying Areas in and around Colombo.

2) Evolving of Appropriate Foundation Types for Construction, particularly for Low-cost Housing in Reclaimed Low Lying Marshy areas.

3) Development of Under-reamed Piling Technique for Sri Lankan Conditions.

4) Engineering Behaviour of Organic and Residual Soils.

In the first project mentioned above, aimed at the geotechnical mapping of low lying areas in and around Colombo, the following objectives were achieved (Senanayake 1986) based on the information gathered from a total of 228 boreholes and augerholes:

i) Identification of low lying areas in and around the city of Colombo which are proposed to be developed in the near future.

ii) Engineering evaluation of the subsurface soil conditions in the study area and identification of characteristic soil profiles.

iii) Formulation of a set of soil series having similar morphology and genetic history to cover the entire project area.

iv) Evaluation of engineering response of each of the soil series to various engineering activities.

In the meantime, a large amount of valuable data has been collected through soil investigations carried out during the past three years. Considering the numerous advantages of a database which can be made available to the planners and developers, NBRO has now undertaken to set up a database using the available personal computer facilities (Ameratunga et al 1989). This database once completed, with further necessary improvements incorporated in, together with the geotechnical maps would be of immense help not only to the planner in his decision making on optimum land use, but also to the developer in preliminary and feasibility studies.

Two important aspects that should be looked into in developing the low lying areas are the drainage conditions and the subsoil conditions. Therefore irrespective of the extent of land, several important steps need to be followed before final decision is made on the type of development. From the geotechnical point of view whatever information that could be obtained on subsoil conditions, nature of any development activities that have already taken place in the vicinity, performance of structures in adjoining areas etc., would prove to be of immense help at the initial stages. Even in an instance when no information is available for a particular site itself, subsoil characteristics in the vicinity could be effectively used in preliminary studies, if such information is available in the database or elsewhere. This information would be often sufficient to decide whether the proposed development is appropriate for the purpose, and if appropriate, to decide on the best layout of the facilities. Also the engineer would be in a position to advise the developer on the appropriate type of foundation for a particular structure or even on how the construction programme could be phased out to accommodate ground improvement where necessary. Such decision making at early stages giving due consideration to the response of the ground would be economical on the long run not only to the developer but to the nation as a whole in keeping with optimum land use.

Once the greenlight is given to the development project, geotechnical maps and database become very useful again in planning detailed investigations. It should be borne in mind that the database could in no way substitute detailed investigations as the latter is considered essential in dealing with low lying areas, where local variations in ground conditions are sometimes apparent. Detailed investigations need to be planned to include all field and laboratory tests in order to obtain all geotechnical information required to design the superstructures, their foundations and any ground improvement methods that are considered necessary.

FIELD INVESTIGATIONS

Field investigations could be of a preliminary nature for decision making on basic planning when no information whatsoever is available from earlier investigations, or could be of detailed nature for the design stage. The extent of detailed investigations required could be determined based on general practice, but in actual execution this would vary with the degree of non-homogeneity of the ground. Often, while in progress, the investigation programme would require changes when new information reveals such necessity.

In Sri Lanka, field borings for subsoil investigations are generally carried out by manual wash boring using bentonite as the flushing medium, mechanised rotary drilling or hand augering. In the marshy areas, considering the accessibility, use of manual methods with light equipment appears to be more appropriate. Collection of continuous samples, disturbed or undisturbed, is very important in uninterrupted classification of subsoil strata and in the determination of engineering properties. Hand augering which caters to both, above appear to be the most suitable exploration method although the depth explorable is limited to about 10 m and difficulties are encountered in advancing through collapsible strata where casings are required. Trial pits and soundings used for general soil investigations are always not applicable to low lying areas. Standard Penetration Testing which provides useful information in sandy soils cannot be used in weak organic soils. Yet, Static Cone Penetration test (also known as Dutch Cone Test) is a very useful and appropriate sounding method in obtaining a continuous strength profile of the usually weak subsoils in marshy areas. By this method of sounding, ground can be investigated efficiently to a reasonable depth of 15 m to 20 m unless encountered by a hard layer which cannot be penetrated. However, soundings should be supplemented with boreholes or augerholes to identify the different strata. The locations for exploration should be identified to cover the whole area to be developed, but at closer intervals especially where high/variable load intensities are anticipated. As a general thumb rule, the spacing between investigation locations may be taken as 10 m to 30 m in the case of subsoil explorations for construction of buildings. In low lying areas where heterogeneity in formations is anticipated, closer spacing are recommended. At least one borehole should be advanced deeper, preferably to the rock or at least to any other suitable hard stratum. This is considered essential because the load from the fill which is usually spread over a large area can influence the subsoils at deeper levels. As a general practice, shallow boreholes should never be terminated in weak layers, but should be advanced through the underlying hard stratum at least through a depth of 3 m. Where preliminary explorations indicate that pile foundation is the Hobson's choice, it is preferable to advance many of the boreholes to confirm the depth to rock or other bearing stratum.

A common application in Sri Lanka as a field test carried out within boreholes or augerholes is the Standard Penetration Test (SPT) for which there are empirical relationships retating its results established with the engineering properties of sandy soils. Its application to cohesive soils is somewhat doubtful at this stage, and in this country, empirical correlations are yet to be made with respect to lateritic soils. However, in the low lying areas, usefulness of SPTs would be limited to grasping strength properties of the fill and other silty or sandy soils found between or underlying the organic soils. Even in the application to gravelly fill, one should be careful of possible erroneous results due to presence of gravel particles.

Vane Shear Test (VST) is probably the most appropriate field test to be used to determine strength characteristics of organic or peaty subsoils with little disturbance to the soil. Even the vane shear strength generally deviates from field strengths with increasing plasticity of the soil (see Bjerrum 1973). Presence of rootlets in organic soils and coarser particles will lead to erroneous results.

SAMPLING AND LABORATORY TESTS

In addition to boring and field tests, it is very important to obtain undisturbed samples (UDS) to determine strength and settlement characteristics of subsoils. UDS should be taken in all soft soils especially within the sphere of influence of loading and in deeper layers where skin friction is involved in pile foundations. In addition, disturbed soil samples should also be collected from respective soil strata, for soil classification purposes.

However, the basic parameters such as natural moisture content and Atterberg limits should be obtained for all organic/peaty soils because they may be useful in many ways during analysis and classification.

Consolidation tests must be carried out on organic/peaty cohesive samples, especially on ones close to the ground surface and within the zone of influence of imposed loads. While consolidated undrained (CU) tests are recommended for organic soils, even unconsolidated undrained (UU) tests may be suitable depending on the circumstances.

Unconfined Compression (UC) test cannot be recommended for very soft soils, however convenient, simple or economical it is.

SOIL PARAMETERS TO BE USED IN FOUNDATION DESIGN

Foundations on low-lying marshy areas are generally placed on a fill which is essentially required not only to keep the ground above flood level, but also to offer a bearing layer with adequate strength above the weak organic strata to support the structures. Therefore to ensure the stiffness of the fill, the adequacy of compaction should be checked with field dry density tests and Proctor Compaction Tests.

As the allowable bearing pressure will in general be determined by the settlement criterion, consolidation parameters become most important. Parameters, such as compression index (C_c) and coefficient of volume change (m_v) will help to calculate the order of settlement expected under the fill load and the building load. Using the coefficient of secondary consolidation (c_α), the creep expected can be obtained. Another important parameter is the coefficient of consolidation c_v, which is used to calculate the rate of settlement. Settlements in the subsoil under the fill load is generally, observed to be at least one order higher than that due to loads imposed by the building. The rate of settlement determined using c_v will indicate the time for completion of primary consolidation, based on which the builder can decide on the earliest time for commencement of the building.

SELECTION OF APPROPRIATE FOUNDATIONS

After completing the analyses of results of field and laboratory investigations, the engineer is in a position to determine the most suitable and economical foundation for the proposed structure. Whether it would be a simple shallow foundation, a stiffened foundation or a deep foundation, will be determined not only on the consideration of the safety but economy as well. In some cases, with the use of ground improvement techniques, and the use of appropriate structural changes to the superstructure, a shallow foundation can be adopted where a pile foundation appears to be the only solution. Stiffened foundations, such as vierendeel girder type of foundations have proved to be suitable where high differential settlements have been expected.

In fact in most low-lying areas, a moderately hard sufficiently thick stratum is found within 6-7 m from the ground surface. For medium to heavily loaded structures, where shallow foundations are found to be inadequate and piles are found to be uneconomical, this stratum can be used for bearing, with the use of under-reamed piles. Under-reamed piles have been used extensively in other countries and in the near future, after the completion of studies done at NBRO, they will be available as an economical alternative foundation type in Sri Lanka too.

SOIL STRUCTURE INTERACTION

One important fact that has emerged from these research is the importance of soil structure interaction in the design of foundations. It is an inevitable mechanism in structures on yielding foundations, such as those supported on compressible ground, which is generally, ignored by most design engineers. The stiffness of the building in relation to the earthfill and the marshy ground has influence on the behaviour of the building and should be taken into account when evolving a criteria for design of foundations.

FOUNDATION DESIGN GUIDELINES AND CODES OF PRACTICE

The findings of the research carried out at NBRO will be made useful to the construction industry by preparation of Design Guidelines and Codes of Practice. The experience gained by engineers in reclaiming the marshy lands and constructing buildings successfully will be included in these. These will be updated often with additional knowledge obtained from the study of the behaviour of foundations in these soils.

The Design Guidelines and Codes of Practice will help to design and construction personnel to select suitable economic foundations in these low lying areas.

The Sri Lanka Standards Institution is setting up Committees at the request of the NBRO to prepare the following:

- (i) Code of Practice for site investigation for foundations.
- (ii) Standard for classification and identification of soils for general engineering purposes.
- (iii) Code of Practice for determination of allowable bearing pressure on shallow foundations.
- (iv) Code of Practice for structural safety of building foundations.
- (v) Code of Practice for design and construction of the under-reamed piles.

FUTURE RESEARCH

Preloading is one of the most successful ground improvement techniques that can be adopted to improve the foundation conditions of the low lying marshy lands. Earth can be used as the preloading material. Once equilibrium is reached in the marshy soil under the preload, the preload is removed and construction of the building commenced. The time-settlement behaviour of the marshy soils in the field is different from the behaviour of small samples of the soil taken from boreholes and tested in laboratory consolidometers. Hence, field studies should be carried by building trial embankments as preloads. The behaviour of the ground will be studied by instrumenting the ground with inclinometers, piezometers, tiltmeters and settlement gauges. This study will help in planning out the development of low lying marshy areas long before construction is to be commenced, thus leading to use of economic foundations in this area.

The understanding of the stress-strain and consolidation characteristics of the peaty soils found in the low lying areas is limited. Hence past experience in construction activities in this area plays an important role in deciding on the type of foundation and the allowable bearing pressures. To produce a more rational method of analysis, further research should be carried out to study the engineering behaviour of these peaty soils in the laboratory.

Dynamic consolidation is another ground improvement technique that should be investigated to see how successfully it can be applied to these low lying areas. In many other countries this method has been used to stabilise marshy ground.

Use of short timber piles to support buildings in low lying areas has found favour with some of the Consultants. This short piles called "root piles" have been used in South East Asian countries to support buildings in marshy lands. Research into the behaviour of timber piles in the peaty soils found in the low lying areas should be undertaken.

Study of the behaviour of the marshy ground under the imposed fill, which is generally lateritic soils or sands, should be continued. Wherever possible, instrumentation such as settlement gauges, piezometers etc. should be used to monitor the behaviour. Settlement studies and distress on buildings founded on the fill should be continued and the results obtained can be used to upgrade the design guidelines and the Codes of Practice.

The use of geotextiles and geogrids, in increasing the load carrying capacity of the earth fill on marshy lands and in increasing the rate of settlement of the fill so that construction can be commenced early, is another field that needs study. Use of local materials like coconut fiber as geotextile should be looked into.

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IMPORTANCE OF FILL IN LOW LYING AREAS

N.W. HERATH

1.0 INTRODUCTION

As a result of rapid increase in population and recent development activities taken place in the city of Colombo and suburbs during the last decade, it has been found that almost all the good building sites have been utilized for development work. The lands required for industrial, commercial and residential purposes are presently found mainly in low lying areas where the construction activities are now concentrated. It has been revealed by the geotechnical investigations carried out by various institutions in the past that these areas are marshes which comprise of highly compressible weak peaty soils of varying thickness (from about 0.5 m to 15 m in the upper layers) (Ray and others 1982, Senanayake 1986). These soils generally possess very high natural moisture content, very low shear strength, high organic content, low specific gravity, high void ratio and high compressibility index. The main constituents of these soils are amorphous peat and fibrous peat in varying stages of decomposition, organic silts and organic clays and/or combination of above with or without intrusions of inorganic soils. The ground water table in these areas is almost at the existing ground level or slightly above/below the ground level.

When such low lying areas are selected for development work, it is essential to raise the ground in order to keep the area above the ground water table or flood level. It is the general practice to raise such ground by controlled or uncontrolled fill. In Sri Lanka, the filling work is usually carried out with lateritic soils, sand, quarry dust or hydraulic fill. Of these, lateritic soils are widely used by the developers as it is found in abundance and due to its comparatively low cost. This paper discusses on the importance of fill material in low lying areas in and around Colombo, associated problems in reclamation particularly with laterites and outlines the procedures for effective filling.

2.0 TYPES OF FILL

All man made deposits of natural soils and waste materials are designated as fill. Of these, rubbish fill which composes of every conceivable type of waste material and rip rap fill which composes of boulders are unsuitable to support permanent structures. Generally the soil used for fill material is known as borrow material as it has to be transported from elsewhere. Sand, gravelly sand, quarry dust are suitable as good quality fill material. In hydraulic filling, where large sites are selected for construction purposes, the soil is usually obtained by dredging it from the bottom of the adjacent river, lake, or ocean and placing it at the desired location.

2.1 Settlement due to Filling

When a fill is placed over a marshy area which usually comprises of peat, immediate compression/settlement of the weak subsoils can be noticed probably due to rearranging of weak solid matrix. High excess pore water pressure also builds up instantly in the subsoils and dissipates with time, resulting significant settlement (primary consolidation) of the ground. In addition, these peaty soils exhibit secondary consolidation causing prolonged settlement with constant effective stress. Hence, high order of settlement can be observed even after a long period of placing the fill over a marshy area.

On the other hand, when a fill is placed without proper compaction procedures or in other words; when the material is loosely dumped, a loose state of compaction prevails within the fill. Buildings constructed over such fills produce settlement within the fill itself. Consequently, large differential settlement, which may develop due to heterogeneity of the fill and compressibility of underlying weak subsoils, cause distress to the structures.

If filling and compaction are properly controlled with good quality borrow material, the fill is likely to be stronger and less compressible than most natural consolidated deposits. Such fill can be used as a good base for foundation of light and moderate buildings. However, compression of the underlying weak subsoils due to fill which behaves more or less like a rigid mat cannot be eliminated. If the extent of the fill area is considerably large compared to the thickness of the weak subsoil layers, compression will be one dimensional and vertical. When deciding the final formation level of the fill against floods, due consideration should be given to the ultimate settlement of the ground due to imposed loads from the fill and the structure.

Fill can also be used as an effective tool to improve poor ground by preloading technique. In this process, a surcharge load of fill is placed for a sufficient time above the normal required level of fill. The time required for improvement of the ground can either be predicted using laboratory consolidation curve or be monitored by placing surface/subsurface settlement gauges in the weak layers. Generally the surcharge load should be selected in such a way that it exceeds by far the total load/pressure exerted by the building. Fill is generally placed over a blanket of coarse sand or quarry dust which not only enables faster dissipation of pore pressures, but make filling operations easy. The surcharge fill could be used to improve adjacent areas subsequently.

3.0 COMPACTION OF FILL

Compaction reduces voids within a fill which helps to control subsequent moisture changes, achieve a state of increased unit weight, increase shear strength of the soil, reduce the permeability and makes the compacted soil less susceptible to settlement upon loading. Hence, compaction is one of the most important basic methods of improving loose soils in a fill.

3.1 Selection of Borrow Material

When a borrow material is identified for the purpose of filling its suitability needs to be evaluated. The following tests are recommended to assess the quality of fill ;

i) Particle size distribution
ii) Standard Proctor Compaction

Compaction test or Standard Proctor Compaction Test (BS1377:1948, Test No.9) is a widely used laboratory test to determine the suitability of a soil as a fill material in reclamation and as a road base construction material.

Generally well graded soils with fines (silt and clay size particles) content not exceeding 30% and maximum Standard Proctor density value not less than 1.8 gm/cm^3 can be taken as a good quality fill material. It should also be free from roots, vegetation and other extraneous matters.

3.2 Field Compaction

When a low-lying area is earmarked for filling it is advisable to strip weak and compressible top surface material. However, this can be overlooked if the root system is considerably thick and the water table is at the ground level. Generally filling of the first layer is somewhat difficult as the borrow material tends to sink into the weak pockets found in the upper layers of marsh. Well woven root system often serves as a geotextile preventing intrusion of fill material into the weak subsoils. However, continuous filling should be kept on until a base is formed just above the ground water table. The thickness of the first layer shall not be more than 600mm loose state. As it is difficult to deploy a mechanical roller at this stage, hand ramming is permitted to achieve at least 85% of the Standard Proctor density. If the soils in the upper layers are soft clays, it is advisable to provide a sand blanket (approx. 0.5 m thick) for efficient lateral drainage of pore water from the consolidating layers. Subsequent layers, however, should be compacted by mechanical means to the specified density (varying from 90% to 95% of the Standard Proctor density) as required which usually depends on the end use of the particular site.

3.3 Field Density Control

It is generally advisable to establish Standard Proctor curves of the particular fill material prior to rolling. This enables monitoring of moisture content requirement for efficient compaction except for the first layer found in boggy areas. Generally, moisture content of the fill material at the site should be within the range of ± 2% to 3% of the optimum moisture content of the same material. Filling should be carried out in layers of not more than 250 mm to 300 mm thickness in loose condition. Field density [Sand-Replacement Method (BS 1377:1949,Test No.10A),Core Cutter Method (BS 1377:1948,Test No.10C) or Rubber Balloon Method] tests should be carried out for every 300 m^2 to 500 m^2 of a fill layer. The degree of compaction achieved should be examined before the next layer is laid. If loose state of compaction is indicated, recompaction needs to be carried out in such areas.

To establish appropriate compaction techniques and to expedite quality control and quality assurance procedures nuclear method (non destructive testing method) can also be used. They are used to determine wet density, moisture content, dry density and degree of compaction at the site it self within a few minutes. However, in order to prevent possible radiation hazards by improper handling of such equipment, training of personnel and adequate safety procedures are essentially required.

3.4 Compactometer

Devices for instantly measuring the effect of compaction equipment on soils have been introduced in the past decade and widely accepted. These are known as 'Roller Integrated Compaction Meters'or'Compactometers 'which are mainly used in sites, where large quantities (around 30000 m^3) of fill is required to be placed and compacted daily, in the developed countries. The principle of the system is based on the interaction between acceleration of the vibratory drum and the stiffness of the soil as the roller travels over the ground. The denser or stiffer the soil, the more rebound is generated on impact.

The compactometer gives a dimensionless reading representing relative value of the stiffness, density or bearing capacity of the soil. This dimensionless value of dynamic measurement can be correlated with the results of conventional methods including density tests, plate bearing tests, nuclear methods or dynamic sounding methods.

The biggest advantage of the compactometer is the obtaining of immediate and continuous information on compaction. This results in a minimum number of passes to reach the specified compaction and better guarantee for homogeneous compaction.

3.5 Type of Compaction Plant

The choice of the field compaction plant is based on a comparison of field and laboratory trial compaction of the same soil. Generally for cohesionless soils vibrators are used. For cohesive soils sheepsfoot rollers are considered as the most appropriate. Pneumatic-tyred rollers are suitable for fine grained soils and closely graded sands. For gravel, sands, crushed rock and hardcore, smooth-wheel rollers are preferred. Number of passes of a particular roller required for specified compaction has to be established by trial and error procedure in the field.

In developing very large extents of low-lying areas, it is preferable to fill one layer at a time over the entire area to avoid failures due to lateral movement etc. of the weak subsoils. Compactometer described above which provides rapid information on the quality of compaction may prove to be useful in efficient handling or such filling work.

3.6 Key Factors in Compaction

It can be stated that in order to achieve better compaction at the site the following factors are of paramount importance.

a) Good quality fill material
b) Correct moisture range
c) Optimum lift thickness of fill layer
d) Appropriate compaction plant

4.0 LATERITE AS A FILL MATERIAL

Laterite soils which are found in abundance in most parts of Sri Lanka are widely used as a fill material in land reclamation in addition to their use as a sub-base and base course material in road construction. The well compacted laterites often provide stable ground to support structures.

4.1 Characteristics of Laterites

Lateritic soils are essentially the product of tropical or subtropical weathering of rocks. The constitutents of laterites will generally form array of minerals. However, chief constituents of lateritic soils are oxides of iron (Fe) and aluminum (Al). Minerals found in laterites include both primary and secondary minerals. The primary minerals are those that remained unaltered from the parent rock and secondary minerals are the hydrated Fe and Al oxides and hydrated aluminium silicates formed in the weathering process. Soil layers generally undergo continuous weathering which does not take place in a uniform or orderly manner. For instance, leaching of lateritic soil profiles depends on the subsoil drainage paths, cracks on the ground and such other arbitrary factors. Consequently, a weathered lateritic soil profile is hetrogeneous and its engineering properties are erratic. In a vertical profile although the entire soil mass is identified as lateritic soils from a civil engineering point of view, it is important to recognise the fact that the profile consists of different types of soils classified as clayey sand (SC), silty sand (SM), clayey silt (ML), silty gravel (GM) and high plasticity clay (CH) and weathered rock. All these soil types exhibit vastly varying geotechnical characteristics. When a quarry or borrow pit is selected as source for fill it may consists of soil types which are even undesirable for compaction.

4.2 Compaction of Laterites

Some of the prominent properties of compacted lateritic soils are the high strength, hardnening on exposure to air and susceptibility to change in properties due to wetting and drying. However, compaction characteristics of lateritic soils are open to many questions. Reuse of specimens of soils instead of fresh specimens to obtain the Proctor curve appears to give a different maximum dry density and an optimum moisture content for some lateritic soils obtained from quarries in Colombo region (Samarasinghe & Herath 1986). This may be caused due to the progressive breaking down of soil particles under the impact of rammer. Very often these laterites get crushed or pulverized in the field too due to rolling or traffic.

Breakdown of coarse particles which improves the grading characteristics may give rise to a higher maximum dry density and lower optimum moisture content. On the other hand, a soil which becomes poorer in grading due to recompaction may give rise to lower maximum dry density values and a higher optimum moisture content. As the grading characteristics and quality of fines in a particular lateritic soil determine the compaction characteristics, the influence of fresh and reused samples on the compaction characteristics of the soil is therefore mainly governed by the strength of coarse aggregates and their subsequent break down. Gidigasu (1970) states that the degradation of coarse particles on compaction of some laterites is influenced by many factors such as parent rock type, genetic origin, position of soils in the vertical profile, maturity and degree of weathering.

However, by carefully selecting, laterites can be used as a fill material effectively. The following specifications can be used as a typical guide:

a) Well graded gravelly sand soil mixture with Gravel and Sand (particle size greater than 75) = 70% to 85%

Silt and Clay (particle size smaller than 75)= 15% to 30%

b) Maximum dry density at optimum moisture content (Standard Proctor density) shall not be less than 1.8 gm/cm^3.

c) The borrow material shall be free of vegetation, roots and such other deleterious material and aggregates should not be fragile upon rolling or ramming.

d) Maximum size of particle shall not exceed 120 mm.

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DESIGN METHODS INCORPORATING SOIL STRUCTURE INTERACTION AND STRUCTURAL CONSIDERATIONS

B.L. TENNEKOON AND A. RAVISKANTHAN

The soil structure interaction studies have resulted in

(i) the formulation of a new design method for foundations based on limiting deformation criteria; and

(ii) the evolution of guidelines for the planning and design of structures in compressible ground.

Both these aspects are covered in this paper.

PART I

DESIGN OF FOUNDATIONS BASED ON LIMITING DEFORMATION CRITERIA

1.0 INTRODUCTION

In the conventional method of design, structures are designed based on the loads that they have to carry but ignoring the settlements of the foundations. Since most structures do settle, they affect the bending moments and shear forces set up in the structure, and this is usually referred to as soil structure interaction. Sophisticated methods of analysis using large storage, high speed computers have been developed to analyse such structures. Examples of these are the 3-D finite element analysis of KING AND CHANDRASEKARAN (1975), MAJID AND CUNNELL (1976); the use of elastic half space theory of FRASER AND WARDLE (1976); etc. However, the subject of soil structure interaction is very complex involving the idealization of both the structure and the soil. The uncertainties involving the overall stiffness of a structrure, the variations in structural loads, the variations in soil geometry and soil property etc. indicate that however accurate the method of analysis may be, it still requires judgement on the part of the designer to obtain realistic solutions. In this context, simple methods of analysis also have their place in foundation design.

This paper presents simple design methods for the design of foundations considering limiting deformation criteria. Several structures have been taken as case studies from the low lying areas of Colombo, as they have undergone relatively large settlements.

2.0 LIMITING DEFORMATION CRITERIA

It is well known that cracking in buildings can be controlled by limiting the differential settlements in the structure. But in actual design, the current practice is to limit the total settlement assuming that, by this means the differential settlements can be kept to acceptable values. For example, LEONARDS (1987) recommends limiting the total settlement to 50mm in sands and 75 mm in clays. However, there are many buildings in very compressible ground where the total settlements are in excess of these limiting total settlements and still show no signs of distress. In such cases it would be advantageous to develop design methods which are based on limiting differential settlements rather than the total settlements.

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

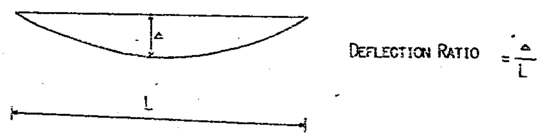


FIG 1a - DEFINITION OF DEFLECTION RATIO FOR LOAD BEARING WALL STRUCTURES

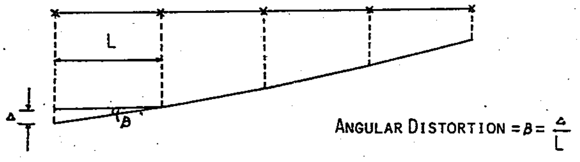


FIG 1b - DEFINITION OF ANGULAR DISTORTION FOR FRAMED STRUCTURES

TENNEKOON ET AL. (1988) have reported on a study of settlements carried out on several structures in the low lying areas of Colombo, and they find that the limiting deformation criterion to prevent architectural damage; i.e. for the onset of visible cracking in plaster, are:

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

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

3.0 DESIGN METHOD FOR FOUNDATIONS OF LOAD BEARING WALL STRUCTURES

In the currently used design method for the foundations of load bearing wall structures,

(i) the width of the foundation is determined from consideration of allowable bearing capacity;
(ii) the transverse reinforcement (for RC foundations) is determined based on the transverse bending moment; and
(iii) the thickness of the foundation is determined by considerations of shear.

However, there is no theoretical method for determining the longitudinal bending moments. In this section, a method is proposed for determining the bending characteristics in the longitudinal direction.

A typical loading system for the foundation of a load bearing wall and the corresponding deformation pattern is shown in Fig. 2a.

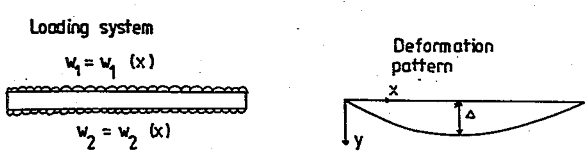


Fig. 2a - Loading system and Deformation pattern for load bearing wall

$w_1 = w_1$ (x) is the load due to the superstructure; and

$w_2 = w_2$ (x) is the load due to the ground reaction.

If $w_1 = w_2$, then there is no net load on the foundation at any point; there is no longitudinal deformation of the foundation; and hence there will be no longitudinal bending moments set up.

But the experimental observations show that the deformation of the foundation is as shown in Fig.1a, i.e. in general, $w_1 \neq w_2$. It is proposed that an alternate loading system be first obtained which will,

(i) give a similar deformation pattern as that of the foundation of the load bearing wall; and

(ii) be simple to analyse for bending moment and shear forces.

Such a system is shown in Fig.2b where a simply supported beam carries a uniformly distributed load w.

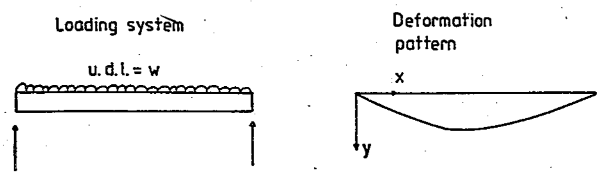


Fig. 2b - Loading system and Deformation pattern for simply supported beam with uniformly distributed load

The first step is to determine the magnitude of w.

It is well known that the maximum deflection for the system shown in Fig. 2b is

$$\Delta = \frac{5wL^4}{384EI} \qquad \text{Eq (1)}$$

The limiting deformation criterion for load bearing wall structures is

$$\frac{\Delta}{L} = \frac{1}{2750} \qquad \text{Eq (2)}$$

Combining Equations (1) and (2) gives

$$w = 0.0279 \frac{EI}{L^3}$$

This equivalent load w is used to compute

(i) the maximum bending moment = $wL^2/8$ and

(ii) the maximum shear force = $wL/2$.

It is proposed that the foundation be designed to carry this bending moment and shear force along the longitudinal direction.

3.1 Application to the Design of Inverted -T Foundations

The cross section of the inverted -T foundation used for a housing scheme in Mattakkuliya is shown in Fig. 3a. The application of the proposed design method is shown in Appendix Ia.

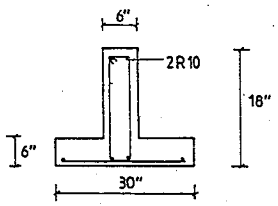


Fig. 3a - Cross section of Inverted T - foundation used for Housing scheme at Mattakkuliya

The results show that the empirically designed beam has a moment of resistance of 7.97 kN m whereas the bending moment which may be expected at the onset of cracking is 9.69 kN m Therefore, according to the proposed method, the section is slightly under-reinforced for carrying bending moment. In the case of shear, it is found that the concrete alone is able to carry the shear force and hence only nominal stirrups need to be provided.

3.2 Application to the Design of Vierendeel Foundation Systems

During the turn of the century, a rectangular frame without diagonals was developed by a Belgian engineer for use in bridge structures, and it was named the 'vierendeel bridge girder' after its inventor. In this paper, foundations which consist of rectangular frames infilled with brickwork, blockwork or masonry rubble, are referred to as vierendeel foundations.

Several authors, SMITH AND CARTER (1969); MAINSTONE (1974); have suggested that when analysing framed structures with infill panels, the structures can be analysed as a frame with the infill being considered an equivalent diagonal compressive strut.

A similar concept is used when designing reinforced concrete beams for shear. In this paper it is proposed that the vierendeel foundations system be analysed for longitudinal bending using the method developed in this section, with the infill being designed to carry the shear. The stub columns can then be placed at nominal spacings similar to the shear reinforcements of RC beams.

The cross section of the vierendeel foundation system using 6 inch blockwork for a housing scheme in Mattakkuliya is shown in Fig. 3b. The application of the proposed design method is shown in Appendix Ib. The stiffness,(EI) of the vierendeel foundation is difficult to estimate, but it must lie between the limits when full composite action is assumed, and when the stiffening effect due to concrete is neglected. It was found that the use of the higher value of (EI) results in very high bending moments and consequently the design sections would have become much larger than those being currently used. Therefore, in this computation the equivalent load w has been determined using the lower limit for (EI).

The results show that the empirically designed beam has a moment of resistance of 36.4 kN m whereas the bending moment which may be expected at the onset of cracking is 13.13 kN m.

Therefore, according to the proposed method, this section is greatly over-reinforced to carry the bending moment. It is also found that the blockwork alone can take the shear, and hence the stub columns can be placed at spacings approximately equal to the lever arm of the vierendeel girder.

A second application of the method has been carried out for a private house in Rajagiriya where the vierendeel foundation system shown in Fig. 3c has been used.

The application of the design method is shown in Appendix Ic. The results show that the beam has a moment of resistance of 120.9 kN m whereas the bending moment which may be expected at the onset of cracking is 133.59 kN m. Therefore, the section is only slightly under-reinforced to carry the bending moment. It is also found that the rubble masonry alone can take the shear, and hence the stub columns need to be placed at spacings of approximately lever arm length of the vierendeel beam.

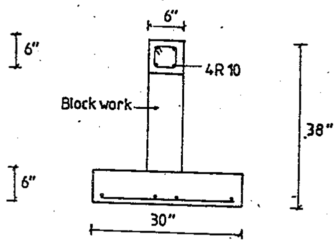


Fig.3b- Cross section of Vierendeel foundation used for Housing scheme at Mattakkuliya

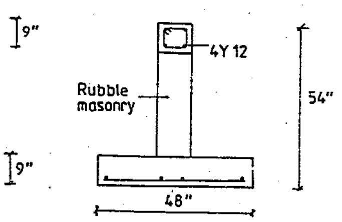
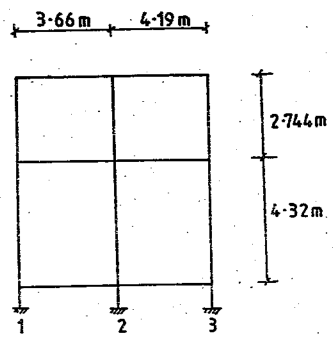


Fig. 3c- Cross section of Vierendeel foundation used for private house at Rajagiriya

4.0 DESIGN METHOD FOR FRAMED STRUCTURES INCORPORATING SETTLEMENTS

Most methods of frame analysis are carried out ignoring the settlement of the foundations. One method of analysis which can be easily adapted to incorporate settlements is the matrix method of analysis. The computer programme MICROFEAP enables such an analysis to be carried out.

The limiting deformation criterion for framed structures was established as an angular distortion of 1/300. The effect of this limiting angular distortion on the bending moments developed in the frame was studied by analysing one of the simple structural forms used in the study. This is shown in Fig. 4 and the frame was analysed by introducing the theoretical settlements shown in the figure, but without consideration of the loads acting on the structure. Two cases were considered. In the first case the angular distortions were in the 'same sense', and in the second case the angular distortions were in 'opposite senses'. It is found that although the same angular distortion has been introduced, the bending moments developed in the frame are very different.



Case (i)

column Node	1	2	3
settlement (mm)	0	12	24

Case (ii)

column Node	1	2	3
settlement (mm)	0	12	0

Fig. 4 Imposed settlements for structure of private house at Rajagiriya to study effects of settlements

For case (i), maximum bending moment = 2.235 kN m
For case (ii), maximum bending moment = 73.367 kN m
i.e. when the angular distortions are in the 'same sense', the bending moments developed are very small compared to when the angular distortions are in 'opposite senses'. Stated differently, it appears that the deformation introduced in case (i) is similar to rigid body type rotation for the structure. Therefore, the angular distortion is not a very suitable parameter for use in design, and an alternative limiting deformation criterion would be required for selecting the settlements to be imposed for the analysis of framed structures.

4.1 An Alternative Limit Deformation Parameter for Framed Structures

Settlement measurements for a number of structures have been carried out by the NBRO as described by TENNEKOON ET AL. (1988). The settlements measured at the column points along the long axis of a building at Peliyagoda are shown in Fig. 5a. The differential settlement pattern after correcting for 'rigid body rotation' is shown in the same figure. A similar analysis carried out for Go-downs at Orugodawatta, and the Open University- Block 1 at Nawala are shown in Figs. 5b and 5c respectively.

It is observed that the differential settlements after correcting for 'rigid body rotation' is similar to the bowl shaped deformation pattern of the foundations of load bearing walls. However, this pattern of differential settlement is not symmetrical about the centre line of the foundation. This may be partly due to the errors in the measurement of these differential settlements which are found to be very small.

It is proposed that the differential settlements of framed structures after correcting for 'rigid body rotation' be assumed to be symmetrical about the centre line and defined by the ratio (Δ/L), where

Δ = maximum differential settlement after correcting for 'rigid body rotation', and

L = length of the building in the direction in which bending is considered

The ratio of (Δ/L) for the buildings analysed are given in Table 1. Although the data available is limited, it is proposed that an alternative limit deformation parameter for the onset of cracking be defined as

$$(\Delta/L) = 1/2750$$

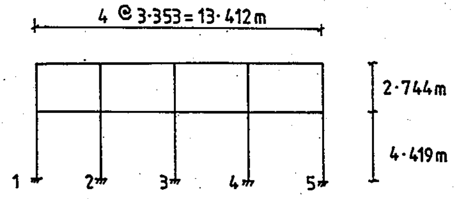
4.2 Effect of Settlement on the Bending Moment Diagram

The effect of settlement on the bending moment diagram for framed structures has been studied using the computer programme MICROFEAP. The building at Peliyagoda has been taken as a typical example, and six different cases of imposed settlement have been considered.

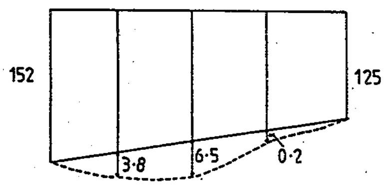
Building	(Δ/L)	β	State of building
1. Building at Peliyagoda	1/2063	1/258	Showing distress
2. Go-downs at Orugodawatta	1/13854	1/762	No distress
3. Open University Block 1 at Nawala	1/3214	1/818	No distress

Note : Subsequent measurements at the Open University Block 1 showed that the angular distortion β reached a value of 1/309, and cracks had appeared.

Table 1 Computation of deformation parameters (Δ/L) and β for framed structures.



column Node	1	2	3	4	5
p (mm)	152	149	145	132	125
Δ (mm)	0	3·8	6·5	0·2	0



$\Delta = 6{\cdot}5$ mm , $L = 13{\cdot}412$ m

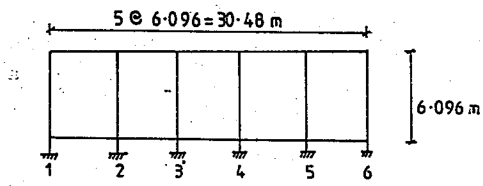
$\frac{\Delta}{L} = \frac{1}{2063}$, $\beta = \frac{1}{258}$

Building is showing distress

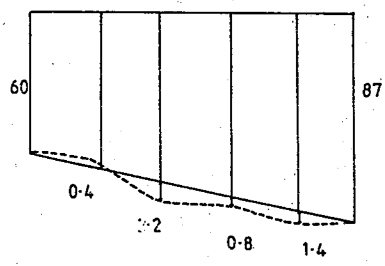
Fig. 5a - Measured settlement of Building at Peliyagoda

In the first case, the structure was analysed assuming that the foundations do not settle. The bending moment diagram for this case is shown as the full line in Fig. 6.

In the second case, the structure was analysed for the loads and imposed foundation settlements corresponding to the 'rigid body type displacement' shown in Fig. 6. The bending moment diagram for this case also is shown in Fig. 6. It is seen that the maximum hogging bending moment has increased from 12.756 kN m to 14.539 kN m; i.e. an increase of 14.0%.



column Node	1	2	3	4	5	6
p (mm)	60	65	73	77	83	87
Δ (mm)	0	-0·4	2·2	0·8	1·4	0



$\Delta = 2{\cdot}2$ mm ; $L = 30{\cdot}48$ m

$\frac{\Delta}{L} = \frac{1}{13{,}854}$; $\beta = \frac{1}{762}$

Building is not showing distress

Fig. 5b - Measured settlement at Go-downs in Orugodawatte

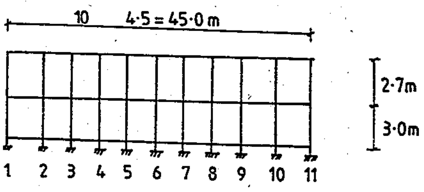
In the third case, the structure was analysed for the loads and imposed foundation settlements corresponding to the actually measured settlements of the foundations. It was found that the maximum hogging bending moment increased from 12.756 kN m to 26.613 kN m; i.e. an increase of 108.6%. However, as discussed previously, the differential settlements after allowing for 'rigid body rotation' are non-symmetrical. It was also mentioned that these differential settlements being small, the errors in settlement measurement may partly account for the non-symmetry which in turn could cause a very large increase in the bending moment.

When this maximum differential settlement of 6.5 mm (after correction) was measured, the building showed distress. It had undergone an angular distortion of 1/258 and the (Δ/L) ratio was 1/2063. As mentioned previously, if it is assumed that the limiting deformation criteria for framed structures is a (Δ/L) ratio of 1/2750, then the maximum differential settlement Δ (after correction) becomes 4.88 mm. Hence, in the fifth case, the structure was analysed for the loads and imposed foundation settlements corresponding to a (Δ/L) ratio of 1/2750. For this case it was found that the maximum bending moment increased from 12.756 kN m to 16.184 kN m; i.e. an increase of 26.9%.

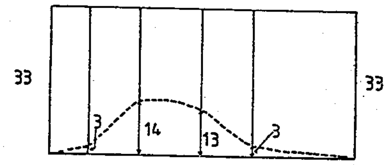
Therefore, in the fourth case, the structure was analysed for the loads and imposed foundation settlements corresponding to a symmetrical distribution of the differential settlement after allowing for 'rigid body rotation', with the maximum differential settlement Δ (after correction) being 6.5 mm as before. It was then found that the maximum hogging bending moment increased from 12.756 kN m to 17.603 kN m ; i.e. an increase of only 38.0%.

In the sixth case analysed, the structure was analysed for the loads and imposed foundation settlements corresponding to the sum of the 'rigid body type of rotation' as in the second case and the differential settlements as in the fifth case. It was found that the maximum bending moment increased from 12.756 kN m to 17.529 kN m; i.e. an increase of 37.4%.

The magnitude of the rigid body type of rotation of the second case is difficult to estimate because some structures do not show any such rotation; e.g. the Open University - Block 1, shown in Fig. 5c. It is believed that this type of rotation takes place when there are variations in the ground in the direction of bending. Therefore, it is proposed that in the first instance, framed structures be analysed only for the differential settlements corresponding to a (Δ /L) ratio of 1/2750. This would slightly underestimate the maximum bending moment if rotation does take place. In the case of the building at Peliyagoda which has been analysed, the neglection of this rotation gave a maximum bending moment of 16.184 kN m, whilst the inclusion of the rotation gave a bending moment of 17.529 kN m; i.e. in this case the error is 8.3%.



column Node	1	2	3	4	5	6	7	8	9	10	11
p (mm)	33	30	-	19	-	20	-	30	-	-	33
Δ (mm)	0	-3	-	-14	-	-13	-	-3	-	-	0



Δ = 14 mm ; L = 45·0m ; $\rho_{average}$ = 29 mm

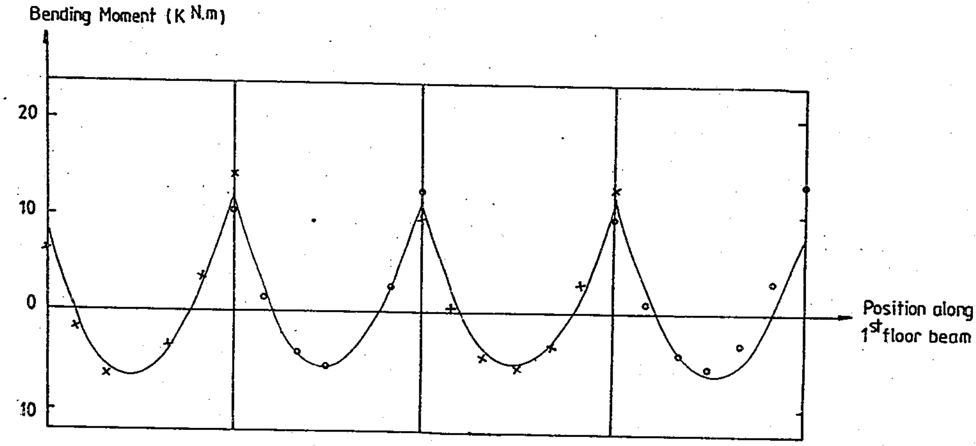
$\frac{\Delta}{L} = \frac{1}{3214}$; $\beta = \frac{1}{818}$

Building is not under distress

(subsequently, $\rho_{average}$ = 45 mm , $\beta = \frac{1}{309}$

Building showed distress)

Fig. 5c - Measured settlement at Open University, Block 1, Nawala



×, o Bending moment diagram for loads and imposed displacements, corresponding to rigid body displacement

—— Bending moment diagram for loads only without displacements

Column Node	1	2	3	4	5
Displacement (mm)	152 (0)	145·2 (0)	138·5 (0)	131·8 (0)	125 (0)

Fig 6 - Bending-moment diagram for Building at Peliyagoda for cases of (i) no settlement (ii) settlement corresponding to rigid body rotation

5.0 CONCLUSIONS

A new design method, based on limiting deformation criteria, has been proposed for the foundations of load bearing walls. This method enables the determination of the longitudinal bending moments and shear forces in strip foundations. The method is first applied to an inverted-T foundation. A case study is taken and it is shown that the theoretically designed foundation compares well with the empirically designed foundation which is behaving satisfactorily. This method also provides a rational basis for the design of the vierendeel foundation which is presently done empirically. Two case studies of structures with vierendeel foundations are taken. On the basis of this method it is found that in one case the foundation has been greatly over reinforced, while in the other case the foundation has been very slightly under-reinforced.

In the case of framed structures, the limiting deformation criterion is given in terms of the angular distortion. However, it is shown that this parameter is not suitable for computing foundation settlements for use in design. It is shown that framed structures show 'rigid body type rotation', and the differential settlements Δ should first be determined after correcting for the 'rigid body rotation'. It is shown that these differential settlements are small, and any errors in their measurement can significantly affect the bending moments developed in the structure. It is therefore proposed that a symmetrical distribution of Δ be used in the frame analysis with a limiting deformation criterion being established in terms of (Δ/L). From the limited data available, this value of (Δ/L) may be taken as 1/2750.

This method of analysis neglects the rigid body type rotation which is found in some structures and not in others. It is believed that this type of rotation takes place as a result of variations in the ground conditions in the direction of bending, and it is, therefore, difficult to estimate. However, in the building at Peliyagoda in which the maximum measured total settlement was 152 mm, and the difference between the total settlements (in the direction of bending) was 27 mm, the error in the maximum bending moment by neglecting tilting was only 8.3%.

6.0 ACKNOWLEDGMENTS

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Appendix Ia

Design of inverted-T foundation for housing scheme at Mattakkuliya

Equivalent load w is given by

$$w = 0.0279 \frac{EI}{L^3}$$

For the blocks of houses analysed,

$L = 20.879$m
$E = 25$ kN/mm^2
$I = 2320 \times 10^6$ mm^4
$EI = 0.058 \times 10^6$ kN m^2

This gives $w = 0.1778$ kN/m

Maximum bending moment = 9.69 kN m
Maximum shear force = 1.86 kN

For the section shown in Fig. 3a,
Moment of resistance = 7.97 kN m
Shear resistance of concrete only = 36.8 kN

Appendix Ib

Design of vierendeel foundation system for housing scheme at Mattakkuliya

Equivalent load w is given by

$$w = 0.0279 \frac{EI}{L^3}$$

For the blocks of houses analysed,

L = 20.879 m

For computation of EI

(a) If full composite action is assumed,
E for blockwork = 7 kN/mm^2
E for concrete = 25 kN/mm^2
EI = 0.4067 x kN m^2

(b) If stiffening effect due to concrete is neglected,
EI = 0.07868 x 10^6 kN m^2

It is proposed that the lower limit of (EI) be used for computing the equivalent load w.

This gives w = 0.241 kN/m
Maximum bending moment = 13.13 kN m
Maximum shear force = 2.52 kN

For the section shown in Fig. 3b,
Moment of resistance = 36.4 kN m
Shear resistance of blockwork only = 19.8 kN

Appendix Ic

Design of vierendeel foundation system for a private house at Rajagiriya

Equivalent load w is given by $w = 0.0279 \frac{EI}{L^3}$

For the private house studied, L = 9.98 m

For computation of EI

Neglecting stiffening effect due to concrete (as in Appendix Ib)

E for rubble masonry = 5 kN/mm^2
EI = 0.3823 x 10^6 kN m^2

This gives w = 10.73 kN/m
Maximum bending moment = 133.59 kN m
Maximum shear force = 53.5 kN

For the section shown in Fig. 3c,
Moment of resistance = 120.9 kN m
Shear resistance of rubble masonry only = 63 kN

PART II

GUIDELINES FOR THE PLANNING AND DESIGN OF STRUCTURES IN COMPRESSIBLE GROUND

1.0 INTRODUCTION

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

(i) the architect plans the building in accordance with the requirements of the client;

(ii) the architectural plans are handed to the structural engineer who designs the superstructure using the most suitable structural form;

(iii) 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 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 (Δ / L) as shown in Fig.1a, and for framed structures in terms of the angular distortion (β) as shown in Fig.1b. 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 NBRO where the settlement of a large number of buildings in the low lying areas of Colombo were monitored. TENNEKOON, SIVAKUGAN AND LAKSHMAN (1988) have reported these results which show 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.

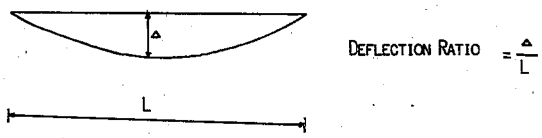


Fig 1a - Definition of Deflection Ratio for Load Bearing Wall Structures

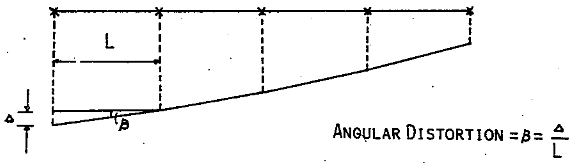


Fig 1b - Definition of Angular Distortion for Framed Structures

3.0 RELATIVE STIFFNESS FACTOR

The avoidance of differential movement in the structure involves a consideration of stiffnesses 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 LAKSHMAN (1988) report on a study made on the overall stiffness of structures in the low lying areas of 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.

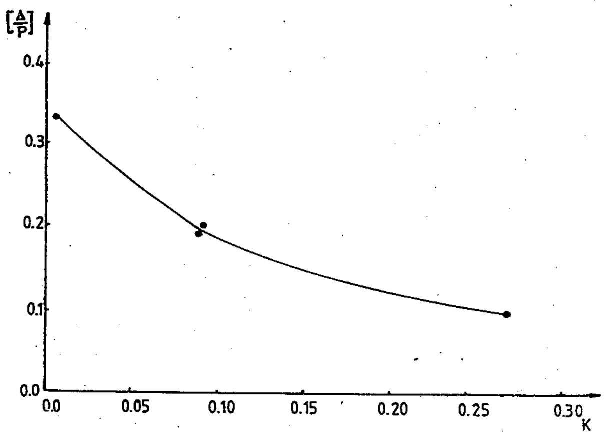
4.0 FOUNDATION DESIGN USING SOIL STRUCTURE INTERACTION

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 superstructure 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, the foundation and the soil. The accuracy of the results depend on how good the models are. In such a situation, experimentally obtained results sometimes give better answers.

Using the results of the settlements measured in the buildings of the low lying areas of Colombo, TENNEKOON, SENANAYAKE AND AMERATUNGA (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 (i) framed structures and (ii) load bearing wall structures are given in Figs. 2a and 2b respectively.

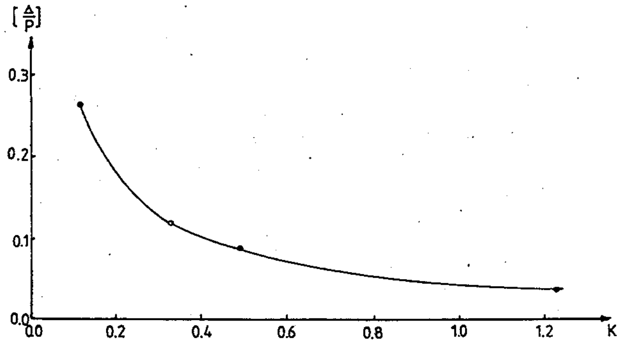


K = Theoretically determined Relative stiffness Factor

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

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

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 = 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. 2b - Relationship between $[\frac{\Delta}{P}]$ and K at failure, assumed to occur at $\frac{\Delta}{L} = \frac{1}{2750}$, for load bearing wall structures

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 (Δ / ρ) 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 Δ for 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 $\Delta / \rho \doteq 0.1$ 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, $\Delta / \rho \doteq 0.035$ 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 Δ/ρ = 0.1 for framed structures and Δ/ρ = 0.35 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:

(i) Reduce 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.

(ii) 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 as it should be 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 brick walls 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:

(i) considering the plan shape of the structure, a simple compact structural unit is better able to withstand differential settlements than an elongated shape;

(ii) the sizes and placings of doors and windows are important;

(iii) 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 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.

Block No.	Range of Settlement (mm)	Average ρ (mm)	Δ (mm)
B	44 - 86	82	1.5
C	74 - 157	115	3
D	153 - 230	191	4
N	70 - 107	92	2.5

Table 1

Settlement recorded in the Nawagampura Housing Scheme where Masonry Foundations have been used.

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.

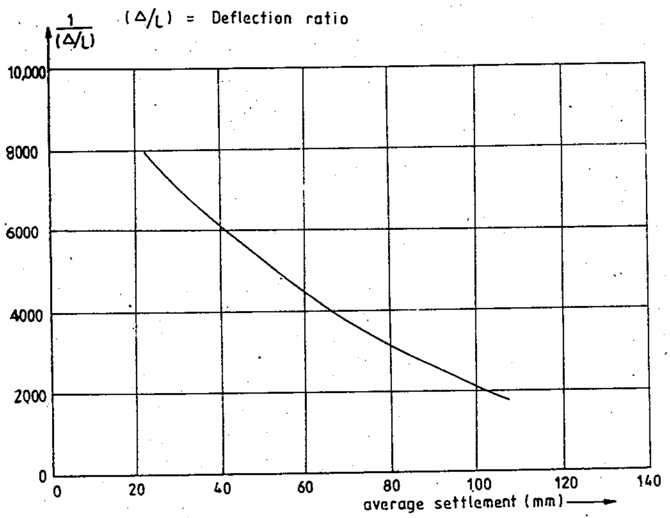


Fig.3- Relationship between deflection ratio & average settlement for blocks in Nawagampura Housing Scheme

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 net 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 (1988) have shown that these foundations can be designed rationally based on limiting deformation 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

$$w = 0.0279 \frac{EI}{L^3}$$

where (EI) is the stiffness of the foundation; and L is the length of the foundation in the direction of bending.

6.2 Foundations for Framed Structures

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

(i) individual pad footings for the columns;
(ii) individual pad footings with the columns connected by plinth beams;
(iii) R.C. strip footings;
(iv) 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 only to the overall 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 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 :

(i) it raises the plinth level so that the building is free from flooding during periods of heavy rain;

(ii) it provides a working platform for men and machines;

(iii) it reduces the stresses coming on the compressible layer to an amount within the allowable bearing capacity of the weak soil; and

(iv) 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,

thickness of peat layer = 5.90 m
thickness of lateritic fill = 2.90 m
depth of footing = 0.90 m
footing dimensions = 1.219 m x 1.219 m

Analysis showed that in this case, the foundation stress of 164 kN/m^2 was reduced to 8.2 kN/m^2 at the surface of the peat layer because of the presence of the fill.

7.2 Settlement 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, than cracking may occur.

At one of the sites which was studied, 1.5 m of fill was placed over an existing fill of 3 m 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 225 mm due to the newly placed fill. As predicted, large settlements did take place and the structure 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 size 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 (Δ/ρ) at failure and the Relative Stiffness Factor (K) for (i) framed structures, and (ii) load bearing wall structures (Δ is the differential settlement and ρ is the total settlement). These results show that

(a) for framed structures, $\frac{\Delta}{\rho} \doteq 0.1$ 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, $\frac{\Delta}{\rho} \doteq 0.035$ 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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APPROPRIATE FOUNDATION TECHNIQUES FOR LOW LYING AREAS

J.J P. AMERATUNGA AND K.S. SENANAYAKE

1.0 INTRODUCTION

The rapid development that have taken place during the past decade or two necessitated the development of low lying marshy areas in and around Colombo, which were hitherto considered not suitable for building purposes. This was because most of the highlands in the region had already been built-up in the past. Development of marshy areas pose many problems to the planners, developers and the engineers. Majority of these are geotechnical problems owing to the extremely poor subsoil conditions prevailing in the low lying areas. One such problem is associated with the foundations for buildings constructed on reclaimed land.

For a particular building project to be economical, the overall cost of not only the superstructure, but also the foundations and any ground improvement work involved should be minimised. But on the other hand, an economical foundation should also be stable and safe enough to meet the functional requirements of the building. Therefore, optimising between economy and safety of a structure depends upon the optimisation of foundation, and this is a big challenge to the geotechnical engineer as well as the structural engineer.

In this paper, an attempt is made to evaluate foundation types and techniques available to the engineer and their appropriateness for the conditions in reclaimed low lying marshy areas.

2.0 A CONCEPT OF APPROPRIATENESS OF FOUNDATIONS

Appropriateness of anything or any action can vary depending on the circumstances and conditions under which the appropriateness is being considered. As far as foundations are concerned, there are two important aspects surrounding appropriateness. They are the safety and economy.

Safety of a structure is of paramount importance and it depends on the ability of its foundations to withstand;

1) shear failure of the ground, and
2) excessive deformations due to ground settlement.

It is seen that the type of structures proposed in the reclaimed marshy areas, which are anyway to be supported over a fill of a reasonable strength, are generally safe against shear failure of the ground. Therefore the major criterion for appropriateness centers on excessive deformation.

It is a generally known fact that it is the differential settlement and not the total settlement that influences distress in a building. Concept of rigidity in a structure becomes important here, since rigidity helps to minimise such differential settlement by converting it to more or less evenly distributed settlement. However, introducing high rigidity in to a structure or a foundation would involve higher expenditure on additional material and construction requirements. Here it should not be forgotten that understanding of the interaction among the superstructure, foundation and the ground, and proper utilisation of the characteristics of their individual and combined behavior will help in arriving at appropriate solutions.

It is also important to note that a certain amount of differential settlement could be tolerated by the structure before yielding any distress. Tennekoon (1989) suggested limiting deformation criteria for load bearing walled structures and framed structures constructed on reclaimed marshy lands. To make the foundations more economical, they could be designed allowing this differential settlement, the limit criterion set above for the onset of cracking in buildings.

In the case of low cost houses it would be very much economical to use simpler low-cost foundations allowing harmless cracks to an acceptable level, instead of adopting costly foundation techniques with a view to achieve a structure absolutely free from distress. Understanding of the building performance would be useful again in appropriate locating of expansion joints, structural joints and in guiding the anticipated minor cracks along pre-designed grooves, so that cracks instead of becoming an obvious eye-sore, could be aesthetically hidden.

Therefore, although the current topic of discussion is appropriate foundations, it could be seen that the appropriateness of a foundation depends on the proper planning and careful design of other members of the structure, and also on the overall cost of the total structure.

When considering the economical aspects of a foundation or the structure as a whole, the importance of "time" factor is often ignored. Specially in the low lying areas, where buildings are generally subjected to distress due to excessive and prolonged settlements, time factor becomes very important. With appropriate planning, the time available for construction should be made effective to economise on a project. For example, preloading as a ground improvement technique decided at the early stages of planning may turn to be an appropriate solution, which otherwise would only be considered as an additional burden that causes delays in the project. This may also mean that time should be assessed in terms of money and that appropriate decisions should be taken at appropriate time.

3.0 APPROPRIATE FOUNDATION TECHNIQUES

3.1 Significant Features in the Low Lying Areas

In developing the low lying areas, one must essentially consider their significant features before deciding on the foundation type or in fact the structure itself. Features coherent and unique to the area should not be looked at with shear anger, scorn or helplessness, but should be exploited to the maximum advantage.

The most significant features one could think of in the low lying areas are;

1) Low lying nature associated with high ground water table
2) Weak and highly compressible subsoil conditions.

3.1.1 Low lying nature

Low lying areas in and around Colombo city generally exist below 2.0 m MSL contour line, but most parts lie below 0.5 m contour and exist as submerged or water logged lands. Even in the other parts the ground water table is generally high.

3.1.2 Subsoil conditions

Another prominent feature of the low lying areas is the presence of subsoils which are predominantly organic and possess unique physical and engineering properties, far from those of inorganic soils (Ameratunga, et al 1989). The significant features of interest associated with these soils, especially the peaty soils are;

1) the high compressibility and
2) the high permeability.

The very high values of natural moisture content (varying from 50% to 550%) and coefficient of volume compressibility exhibited by these subsoils clearly indicate their susceptibility to undergo very high settlements even under a small load imposed. On the other hand, because of the high coefficients of primary consolidation owing to high permeability, the rate of settlement in the primary stages will be very high. However, the coefficient of secondary consolidation is low, and therefore secondary settlement can take place over prolonged periods.

3.2 Necessity of a Fill

If any development activity is to take place in the low lying areas, the land should be reclaimed to raise the ground well above the water table and flood levels on account of the low lying nature. On the other hand, because the weak and highly compressible nature of the subsoils, it is necessary to provide a safe and stable ground surface for executing the construction activities. This can be achieved by controlled earth filling to a suitable level. However, increased loads due to the fill will cause significant settlement in the underlying subsoils which are already undergoing consolidation under their own weight. On the other hand the fill itself may become a major problem if the filling is not properly done.

3.3 Earth Filling as a Ground Improvement Technique

3.3.1 Ground settlement due to development activities

The main objective of ground improvement in the case of low- lying areas is to increase the strength and to reduce compressibility of the ground supporting the structures and thereby to minimise the cost of foundations and hence the project cost. Before dealing with this essential and inevitable earth filling as a ground improvement technique simple in execution, it would be useful to look at the components of settlement in reclaimed marshy lands due to development activity.

The settlement S_t of structure supported on a reclaimed land will be composed of the following;

$$S_t = S_1 + S_2 + S_3 + S_4 + S_5 \quad ....(1)$$

where,

S_1 : Settlement in the underlying subsoils due to own weight
S_2 : Settlement in the underlying subsoils due to load imposed by the fill
S_3 : Settlement in the underlying subsoils due to load imposed by the structure
S_4 : Settlement in the fill due to own weight
S_5 : Settlement in the fill due to load imposed by the structure.

It could be seen that, if the fill is extremely well compacted, the components S_4 and S_5 become negligible (see Table 1). Again, if the foundation is placed on the fill at an elevation sufficiently high above the interface of the fill and original ground, so that the pressure bulb of the foundation load is practically within the fill, component S_3 too become negligible. Now, S_1 being the natural consolidation of the subsoil undergoing through a slow geological process, S_2 becomes the major and important component of settlement.

Table 1. Total settlement of different fill materials under their own weight.

Type of fill		Total Settlement under its own weight expressed as % of the fill thickness.
Well compacted inorganic fill		0.5 %
Poorly compacted inorganic fill	up to	1 %
Well compacted clay fills		0.5 %
Highly compacted clay fills placed in deep layers		1 % - 2 %
Domestic refuse fill (with controlled topping layer)	about	10 %

The stress imposed in ground due to loads from a foundation of limited width gradually decreases with depth. On the contrary, fill load distributed over a wide extent of ground surface will be effective to the full depth of the underlying subsoils, which is about 20 m at the most.

The settlement in underlying subsoils due to fill load can therefore be expressed by,

$$S_2 = m_v \cdot \delta_f \cdot H_f \cdot H_s \quad \ldots\ldots (2)$$

where,

m_v : Coefficient of volume compressibility of the compressible subsoil

δ_f : Unit weight (or submerged unit weight) of fill

H_f : Height of fill

H_s : Thickness of compressible subsoil

Using typical values for m_v and δ_f, the relationship between settlement and the thickness of weak compressible subsoil can be shown in Fig. 1.

High costs involved in large fill volumes and the excessive settlements anticipated in underlying soil layers due to high fill loads would not always permit filling to a suitable height so that foundation loads could be contained within the fill layer.

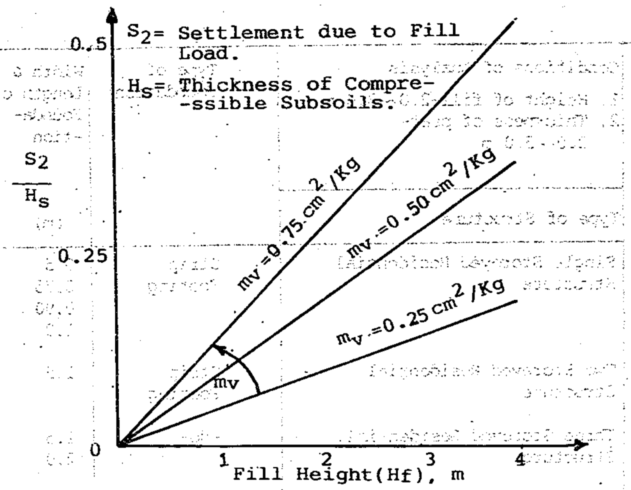

Fig.1. Settlement due to Fill

Looking at the effect of fill again, it is clearly seen in Table 2, reproduced after Senanayake (1986), that the settlement expected in underlying soil due to the fill is in fact at least one order high in most cases than that due to building load. This shows that the contribution due to fill load on the overall settlement of a structure supported on the fill is generally high. Therefore in order to minimise adverse effects on the structure due to high settlements, it would be necessary to delay the commencement of building construction until major part of the settlement is over so that the residual settlement is brought within tolerable limits. In most peaty soils, primary consolidation takes place fast, but since secondary consolidation cannot be ignored it is necessary to closely monitor the progress of settlement in deciding the proper time to commence building activities.

Therefore, the primary appropriate foundation technique in developing low lying areas particularly for low-cost buildings is to ensure;

1) A well compacted fill

2) Adequate thickness of fill to overcome problems due to ground water and floods

3) Adequate depth of well compacted fill below foundation level so that the underlying layers are least affected by foundation loads

4) Adequate time for major part of settlement to take place under the fill load leaving tolerable residual settlement before commencing building construction

5) By monitoring to confirm the rate of settlement has reduced to an acceptable level that building construction could be commenced.

TABLE: 2 - RESULTS OF SETTLEMENT ANALYSIS FOR M-I SUBSERIES AS MODEL

	Conditions of Analysis 1. Height of fill-2.0~3.0 m 2. Thickness of peat-2.0~3.0 m Type of Structure	Type of Foundation	Width & Length of Founda--tion (m)	Depth of Footing Below Fill Level (m)	Maximum Contact Pressure assumed at Foundation. Base (KN/m^2)	Settlement of foundation under Load of Fill & Structure (mm) Range Low High	Settlement of foundation under Load of Structure (mm) Range Low High	Average % Settlement contributed by structure
1.	Single Storeyed Residential Structure	Strip Footing	0.5 0.75 0.90 1.0	0.5 0.5 0.5 0.5	50 33 28 25	260-303 260-301 259-301 259-300	05-14 05-12 04-12 04-11	3 3 3 3
2.	Two Storeyed Residential Structure	Strip Footing	1.5	0.5	43	267-320	12-31	7
3.	Three Storeyed Residential Structure	-do-	1.5 2.0	0.75 0.75	77 58	279-346 277-340	24-57 22-51	13 12
4.	Four Storeyed Residential Structure	-do-	2.0	0.75	83	287-362	32-73	16
5.	Light Industrial Structure with 25 KN/m^2 imposed load on ground floor	Pad Footing	3.0x3.0 3.5x3.5	0.75 0.75	33 31	331-425 328-430	76-136 73-141	28 28
6.	Heavy Industrial Structure with 100 KN/m^2 imposed load on ground floor	-do-	3.0x3.0 3.5x4.0	0.75 0.75	119 113	344-453 343-435	89-164 88-146	32 30
7.	Three Storeyed Office Structure	-do-	2.5x2.5 3.0x3.0	0.75 0.75	221 153	308-413 303-400	53-124 48-111	25 23
8.	Four Storeyed Office Structure	-do-	2.5x2.5 3.0x3.0	0.75 0.75	304 211	322-440 317-427	67-151 62-138	28 27

However, the above ideal conditions cannot be always achieved in practice. If the project is properly planned and carefully managed, perhaps it may be possible to meet at least some of these requirements.

3.3.2 Importance of a well compacted fill

The importance of a well compacted fill needs to be emphasised here in appreciating its role in;

1) providing a hard bearing stratum to support the foundations,

2) providing a bearing stratum which would undergo only negligible settlements under foundation load,

3) providing a stiff mat below the foundation which helps to distribute the load from foundation more or less evenly.

How to construct a well compacted fill is discussed by Herath (1989).

3.3.3 Determination of the thickness of fill

The minimum thickness of the fill should be decided considering two aspects as already discussed, viz. clearance above ground water level or flood level, minimum depth of compacted fill required below foundation level, and the settlement in the ground due to fill load. Generally, it is not a good practice to keep the finished ground surface in different lots within the same area of development, at different levels. Therefore, priority may be given to the clearance above flood level estimated from hydrological studies. In the case of low lying areas in and around Colombo, the final formation level has been decided approximately 30 cm above the 200 year probable flood level (CDLLAD&DB,1980). It has been noticed that this final formation level is mistakenly interpreted as the level at which filling should be terminated. As a result, where allowance had not been made for the prolonged settlement of the ground under fill load, structures already constructed on the fill would be prone to submergence during floods or perennially by ground water. To differentiate between the reclaimed/filled level and the intended level above which the ground surface should be maintained on the long term, Senanayake (1988) has suggested to refer the latter as the Planned Ultimate Formation Level (PUFL).

Now if the difference between PUFL and the existing ground level (before filling) is given by H_u, the thickness (H_r) of fill actually required could be approximately obtained from ;

$$H_r = H_u/(1-m_v.\gamma_f.H_s) \quad .....(3)$$

The above relationship is graphically presented in Fig.2. However, this relationship cannot be used for estimating the fill height when the surface soils are very weak and the fill material intrude into the ground.

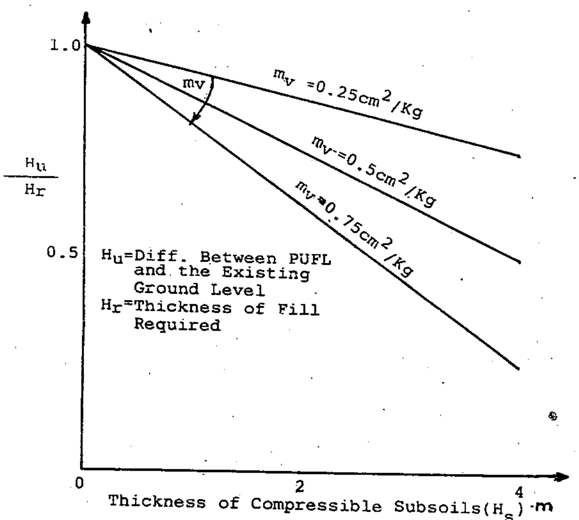

Fig.2. Height of Fill Required

3.3.4 Adequacy of the depth of fill below foundation level

If the depth (D_1) of compacted fill available below the foundation level is large enough to contain the stress bulb of foundation within the well compacted filled layer without shear failure, special techniques may not be necessary. In this case, D_1 should be equal to at least 2.0 to 2.5 times the width (B) of foundation. When the stress bulb protroduces into the underlying soil layer, settlement will take place in the underlying subsoils due to the foundation stresses distributed. This settlement can be estimated based on stress influence factors suggested by Boussinesq or by approximating the stresses to spread uniformly at an angle of 60° with the horizontal. Extreme care would be necessary when the depth of fill below the foundation level is small as shear failure could occur. In this case, an appropriate solution would be to replace the weak underlying soils below the foundations with well compacted fill. Considering presence of ground water, a compacted granular fill would be preferable and the depth of replacement should be equivalent to more than the width of foundation (Fig. 3).

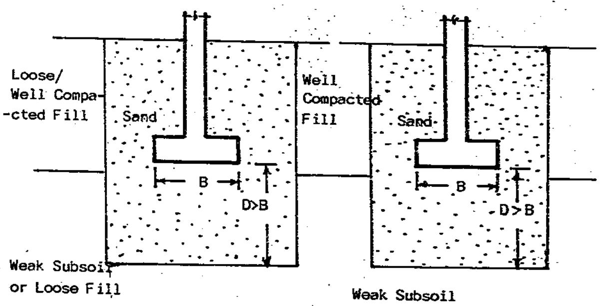

Fig. 3. Foundation on Compacted Granular Fill

3.3.5 Uncontrolled fills

Achieving highest standards in compacted filling is not always possible, as this involves high costs of construction, time and resources for management, even though maintaining high standards will help cut down overall costs on the long term. On the other hand there are situations where construction specifications are relaxed or overlooked by the engineer judging from the importance of a project.

Specially in areas where planned development is not implemented, small plots of marshy lands are observed to have reclaimed without any control whatsoever, using any material, thickness bargaining with domestic waste to industrial refuse.

The artificial ground thus formed is often complex and heterogeneous in its structure, strength and other engineering properties. Loosely dumped wastes or uncontrolled earthfill can pose more geotechnical problems to the engineer than the weak underlying subsoils.

Such situations may call for total replacement or partial replacement of the fill with a controlled fill. One appropriate solution is to place the foundations on a granular fill as discussed under 3.3.4. As alternatives, placing of a sufficiently thick well compacted layer over the existing fill, with or without partial replacement, or improvement of the ground by pressure loading could be considered.

4.0 GROUND IMPROVEMENT TECHNIQUES

4.1 Preloading as a ground improvement Technique

Preloading technique involves loading of the ground to pre-induce before laying of the foundation, a greater part or whole of the ultimate settlement that the ground is expected to experience under the loads imposed by the development activity. It is an appropriate technique for ground improvement especially in the low lying areas where peaty soils which are highly compressible and highly permeable and therefore do not require much time to achieve the anticipated results.

This method is most appropriate in developing long extents of low-lying areas where economically available borrow material could be used as the surcharge material progressively in bulk and moved across using machinery. If the work involved from preloading to the construction of structures is planned carefully, this technique appears to be ideal for improving ground with peaty subsoils which are generally highly compressible and highly permeable, and hence responding fast. By increasing the surcharge load within permissible limits, the consolidation process can be accelerated to achieve the results faster. However, when preloading peaty soils it is important to note that swelling or the rebouncing of the peat layer after removal of surcharge could be high. Although the design of preloading could be done based on theoretical assumptions using the soil parameters, considering the complex behaviour of organic soils, it is an essential and important practice to monitor the field performance in order to decide on when to remove the surcharge load.

The authors have observed that in Sri Lanka, many engineers show apparent reluctance to adopt preloading as a ground improvement technique. This may be due to the lack of documented case studies or due to the fear that the project would be unnecessarily delayed. But on the other hand, it had also been noticed that in some cases the planners have spent several months or years deciding on the appropriate foundations, while costs were soaring and the land awaiting idle.

To minimise cost of preloading and construction time the following methods can be suggested.

1. In large areas, reclamation and preloading can be done progressively. The priority areas of construction should be reclaimed first and preloaded. Once preloading is completed in one area, the surcharge load could be used for reclaiming and preloading the successive area.

2. Use of the reclaimed land for stock piling of heavy items.

3. Instead of reclaiming the entire area of the land, to concentrate reclamation to the building area and to surcharge this area with fill material that is intended to be used for the balance area.

4. After reclaiming the building area to surcharge with construction materials.

4.2 Use of Sand Drains

When it is necessary to further accelerate the consolidation process, vertical drains could be used. Vertical drains, purpose of which is to provide shorter drainage paths in the lateral direction when thickness of consolidating layer is high, may be economical only in large projects. However, since small diameter sand drains placed at closer intervals are more efficient than large diameter ones spaced far apart, installation of small diameter sand drains packed in fibre columns using a simple hand auger may be worth attempting at least on an experimental basis.

Where ground improvement is expected by preloading with or without sand drains, it should be emphasised that a coarse permeable layer (sand or gravel or quarry dust) must be placed in between the existing ground and the filled layer to improve drainage of pore water from the consolidating soil. Granular materials are also suitable in achieving required compaction below water table.

4.3 Other Ground Improvement Methods

4.3.1 Stone/Gravel Columns

Stone or gravel columns installed in poor ground can transmit foundation loads to stronger strata at deeper levels. Further, when combined with preloading, these columns will serve not only as vertical drains but also in forming a composite ground.

4.3.2 Surface Solidification

Ground can be improved by surface solidification with cement, lime or other chemicals, depending on the soil type. However, these methods would be very costly when considering the effectiveness in applying to organic soils as found in marshy areas and the special equipment that would be required.

4.3.3 Geotextiles

With the installation of materials such as geotextiles, sheets, fagots, nets, ropes or bamboo rafts poor ground can be reinforced by effectively utilising the combined strength of ground and these materials. Geotextiles/fagots are useful in controlling thickness of fill during filling operations which would eventually help in minimising differential settlement due to fill load.

4.3.4 Dynamic Consolidation

Large settlements in the underlying compressible strata can be curtailed by the densification of these strata. The dynamic consolidation method which was originally used for densification of loose sandy strata has been successfully used on organic soils. In this technique, a heavy weight is dropped on the ground from a considerable height, systematically over the area to be improved, e.g. on a grid pattern. The craters formed by the impact are then filled with quality soils and compaction process is repeated until the ground attains required level of improvement. This method is recommended for development of large areas and the equipment required are rather simple.

5.0 APPROPRIATE FOUNDATION TYPES

5.1 Foundation Types Used in the Low-Lying Areas

In the research project undertaken by NBRO, it was planned to monitor a large number of existing (old) and newly constructed buildings and to analyse their performance with a view to identify the foundation types appropriate under different conditions. However, the number of existing buildings that were constructed several years ago was limited and it was observed that most of them were apparently constructed close to the peripheries of marshy lands where ground conditions are not so bad. Such buildings supported on conventional shallow foundations were observed to have performed satisfactorily, whereas a few buildings located in areas with much poor ground conditions have shown minor distress.

On the other hand, a large number of new buildings have been constructed in the reclaimed marshy areas using different types of foundations including conventional and specially innovated types. These foundation types and their performance had been discussed (Sivakugan et al 1989, Tennekoon, 1989, Tennekoon and Raviskanthan, 1989).

The findings of the research project with respect to appropriateness of these foundation types are summarised in the following section.

5.2 Appropriateness of Foundations

Foundations types considered in the NBRO study includes the following:

i) Conventional Strip Footings

- a) Rubble/Brick Masonry
- b) Rubble/Brick Masonry with RC Strips

ii) Stiffened Strip Foundations

- a) RC Inverted T' Type
- b) Vierendeel Girder Type

iii) Raft Foundations

- a) Rigid Raft
- b) Flexible Raft

iv) Pad Foundations

- a) Isolated Pad Footings
- b) Pad Footings with connected Plinth Beams

5.2.1 Strip Foundations

Strip foundations are generally used for load bearing walled structures or framed structures where it is necessary to increase stiffness of the structure as a whole. Rubble or brick masonry foundations are used for load bearing structures where settlements expected during the life time of the structure is considerably small. It is generally believed that strip footings consisting rubble/brick masonry with RC strip at the bottom help to tolerate the differential settlements more than rubble masonry footings without RC strips. Tennekoon and Raviskanthan (1989) have shown from observations of settlements of buildings in low lying areas, that this notion has no sound rationale. In fact, as the deflection ratio for onset of cracking is the same for both the above types, it is concluded that masonry foundations with or without RC strip perform similarly.

Since no special technology is required in their construction which could be done at a relatively low cost, rubble/brick masonry foundations have been conventionally used in single storeyed construction. Considering the advantages and performance, rubble/brick masonry footings, without the RC strip can be taken as more appropriate where strip footings could be used, specially for light loaded buildings.

5.2.2 Stiffened Foundations

Reinforced concrete inverted T foundation and the vierendeel foundation are stiffened foundations which are often used in weak soils with low bearing capacity. The reinforced base slab provides increased bearing area required. Although the transverse reinforcements are designed for cantilever action, there is no sound theoretical method to design the longitudinal reinforcement. Tennekoon & Raviskanthan (1989) have shown a theoretical approach to the design of a vierendeel girder. Stiffness of these foundations which can be effectively enhanced by increasing the height of the web, is generally much greater than that of masonry foundations with or without the RC strip.

However, Tennekoon (1989) has pointed out that it is the stiffness of the total structure that matters and not the stiffness of the foundation alone. In the case of load bearing structures as the contribution from the foundation to the total stiffness is small and hence the net effect of providing a stiffened foundation is not very significant. However, in structures with load bearing walls where many openings are included, the contribution from a stiffened foundations is desirable.

In framed structures, vierendeel foundation is a good method of providing stiffness to the structure, considering that the superstructure tend to be less stiff because of large openings.

Tennekoon and Raviskanthan (1989) has also compared the stiffness provided by two empirically designed foundations, i.e. vierendeel and inverted -T', for construction of houses within the same site. It was shown that the vierendeel foundation provides a higher relative stiffness factor than an inverted -T foundation.

5.2.3. Pad Footings

Building in low lying areas are usually constructed on a 1 m - 2 m thick fill over the poor ground. If the fill is generally compacted to the specifications and thus offers adequate strength to support the foundation, pad footings could be designed such that the influence zone does not extend deep into the underlying poor ground. For square or circular pad footings, the stress increase at a depth of 2B, where B is the width of footing, would be about 10% of the stress applied at the foundation level whereas in a strip footing the influence zone is narrowed very much deeper.
By placing pad footings at small spacings, stress distribution could be spread horizontally rather than vertically and therefore the strength of the fill is utilized to the maximum. However, from consideration of stiffness necessary to absorb differential settlements, it is desirable to join all columns by a grid of beams at plinth level. Pad footings are sometimes provided even when the walls are of load bearing type, by providing short columns below the grid of beams.

5.2.4 Raft Foundations

Raft Foundations are used when the bearing pressures should be drastically reduced owing to low bearing strength of the supporting soil or when it is necessary to reduce differential settlements by bridging over variations in subsoils.

Although it is generally possible to increase the bearing capacity of the ground by using a raft, this is counter effective in low lying areas as the weak soils below the fill layer of limited thickness is affected by the foundation load to a greater depth. Therefore, in adopting raft foundations on fills over weak subsoils, it is important to rely mainly on the strength and stiffness of the fill which jointly function as a raft.

Some innovative modifications of raft foundations used in the low lying areas include very thin reinforced floor slabs used alone or jointly with pad footings to support single to two storeyed structures.

5.2.5 Deep Foundations

Heavy structures which are likely to undergo large settlements if supported on shallow foundations, should be supported on suitable bearing strata available at deeper levels. Piles, cylinders etc., can be used for transmitting the foundation loads. Nature of subsoil conditionds in the low lying areas in and around Colombo generally do not permit use of friction piles. Hence end-bearing piles supported on bed rock (generally observed at depths ranging from 15 m - 25 m) appear to be the Hobson's choice of foundation for heavy structures.

In contrast to the deep piles required for heavy structures, under-reamed piles (Thayalan et al, 1989) can be used for supporting moderately loaded structures on shallow, moderately strong inorganic soil strata encountered in certain parts of the low lying areas.

When selecting piles as the appropriate foundation for supporting structures it is important in the design to carefully consider the effect of negative skin friction (Senanayake, 1988) developed on the pile as a result of subsequent settlement of surrounding soil.

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ADAPTABILITY OF UNDER-REAMED PILES FOR LOW LYING AREAS

N. THAYALAN, N.W. HERATH AND K.S. SENANAYAKE

1.0 INTRODUCTION

A major problem encountered in developing low lying areas is the presence of highly compressible, predominantly organic subsoils existing closer to the ground surface. These weak soil strata exist as a single layer or as two layers sandwiched between inorganic soils and occur in thicknesses from about 0.5 m to as high as 12 m. As a result, high order of settlement, which could be detrimental, can be anticipated in structures supported on conventional shallow foundations constructed over reclaimed marshy lands. A common practice to overcome this problem is to support the structures on costly special foundations which could accommodate differential settlements to a greater degree or to support them on pile foundations. However, conventional pile foundations need to be supported on a firm stratum and often as end-bearing piles in the case of low lying areas where the overburden does not offer adequate resistance for friction piles. When suitable bearing stratum, usually rock, is found only at large depths varying between 15 m to 25 m, foundation cost becomes prohibitively high and not commensurable with the cost of superstructure specially in the case of small scale and low- rise buildings with low to moderate loads.

Detailed analyses of the subsoil profiles have shown that in certain parts of the low lying areas, moderately hard and sufficiently thick layers of stiff inorganic clay or medium dense sand are available within a reasonable depth range of 3 m to 7 m which can be considered as a suitable bearing stratum for lightly loaded foundations. Small diameter bored and cast-in-situ piles with under-reamed bulbs formed in such moderately hard layers can be used effectively and economically to support lightly to moderately loaded structures.

Therefore, National Building Research Organisation (NBRO), who was already engaged in active research on the development of low lying areas, especially with a view to evolve appropriate foundations for construction in the reclaimed marshy areas, identified the development of under-reamed piles applicable to local conditions as an important research area. This paper discusses the findings of an ongoing research project undertaken by NBRO with the main objective of developing the technique for installation of under-reamed piles through weak soils particularly in marshy areas.

2.0 DEVELOPMENT OF UNDER-REAMED PILES

Piles with enlarged bases have been widely used in some of the developed countries for more than half a century, specially during the early stages of their industrial development. These were adopted in situations where a suitable bearing layer could not be found closer to the ground surface for shallow foundations and when it was necessary to transmit a considerable foundation load to a bearing stratum located at a moderate depth.

Pedestal piles, Frankie piles, Compressol piles, and Simplex piles etc., are some piling techniques that were used to construct a concrete bulb at the lower end of a reinforced or unreinforced shaft of short piles (Fig.1). A common feature in some of the techniques was that the bulb is formed by compressing the concrete laterally into the ground using a heavy drop hammer to compact the concrete directly or by transmitting the impact energy through a timber or metal shaft. There are also records of a technique where the cavity for the concrete bulb is formed by blasting of an explosive.

The usage of small short piles with enlarged bases gradually reduced when the demand for large scale buildings and structures became greater in the developed countries with economic growth. In fact, the use of large diameter deep piles with under-reamed wide bases for heavily loaded foundations appear to be the current trend in some of the developed countries.

In contrast to the compressed bulb type piles that were used widely in the West and Japan, an under-reamed piling technique that could be manually operated was developed at the Central Building Research Institute in India (CBRI, 1965) as an effective foundation type suitable for ground movement problems in Black Cotton and other expansive soils. Under-reamed piles are now being used widely, and sometimes even indiscriminately, in the developing countries for load bearing in other types of soils too.

The under-reamed piles which are essentially bored cast-in-situ piles, are quite simple to construct using a specially designed under-reaming tool. A hole to the required diameter of the pile shaft, is excavated by augering or by bailing the soil out. The hole in the ground soil is supported using bentonite slurry during excavation operation and thereafter until under-reaming and casting of the pile is completed.

These piles can be designed as single under-reamed or as multi under-reamed having two or more bulbs to achieve increased bearing or pulling resistance. Detailed information on the state of the art of design and installation of under-reamed piles under different soil conditions are dealt in the CBRI publications (CBRI, 1965).

3.0 ADVANTAGES OF UNDER-REAMED PILES

Under-reamed bulbs, which can be provided at desired depths, where substantial bearing or anchorage is available, are useful in two ways. The bulb provides large bearing area at the required depth, and also serves as an anchor and keeps the foundation stable even in the event of any upward drag on the pile. The choice of under-reamed piles over uniform diameter piles have the following added advantages. The under-reamed bulbs are generally designed to have a diameter of about 2.5 times the diameter of the pile stem, thereby increasing the end bearing area by more than 6 times. Therefore material required could be saved by about 500%. Moreover, downward drag developed on the pile due to negative skin friction will be much less on the smaller surface area of the pile stem.

4.0 OUTLINE OF THE RESEARCH PROJECT ON DEVELOPMENT OF UNDER-REAMED PILES

4.1 Objectives and Activities

With the objective of developing under-reamed piling techniques suitable for installation through weak superficial organic subsoils for supporting foundations of low-rise houses or structures, particularly in marshy areas, NBRO launched a research project in 1987. The major activities undertaken to achieve this objective are;

a) Review of the subsoil profiles of low lying marshy areas, the sequence and extent of horizons, and the range of engineering properties.

b) Review of the development activities proposed to be undertaken in low lying marshy areas, structural details, load intensities etc.

c) Review of various under-reamed pile types with regard to size, range, installation details, performance under different conditions etc.

d) Development of special features of installation equipment, as necessary, to suit specific ground conditions and loading details.

e) Installation of prototype piles in different sizes in different soil conditions and assessment of pile behaviour under load.

For this project sponsored by the United Nations Development Programme, NBRO obtained technical services from the Asian Institute of Technology in Bangkok, Thailand.

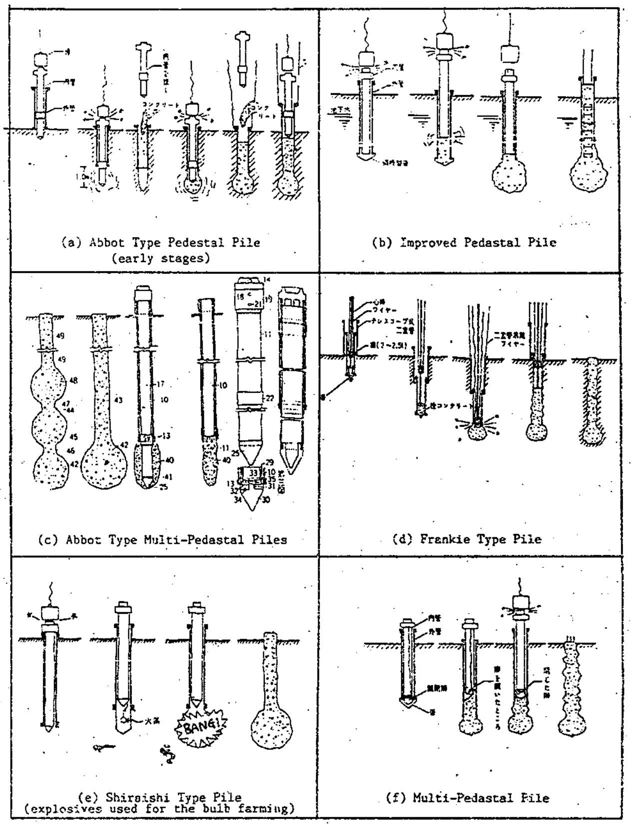

FIG. 1 CONSTRUCTION STAGES IN SOME TECHNIQUES FOR CAST-IN-SITU PILES WITH ENLARGED BASE

4.2 Review of Subsoil Profiles in Low Lying Areas

Information available from a large number of boreholes and auger holes advanced in the low lying areas were carefully studied with a view to identify locations where under-reamed piles could be adopted as an effective foundation type. Data used for this purpose include over 200 boreholes studied in the Geotechnical Mapping of Low Lying Areas in and around Colombo City (Senanayake, 1986), and those documented thereafter under specific consultancy projects and research projects undertaken by NBRO. It was found that about 25% of the boreholes studied exhibit soil profiles where a suitable bearing stratum for the under-reamed bulb could be found within a depth of 3m to 7m from the ground surface. In identifying a suitable bearing stratum for under-reamed piles, the strength of the ground for several meters below the potential bearing level should be duly considered, in order to keep the stress influence within the hard stratum without affecting any underlying weak layers. Piles with bulb diameters in the range of about 600 mm to 1000 mm are considered appropriate for the order of loads transmitted from low-rise buildings. Therefore, soil layers where moderately hard or hard ground (SPT N-values prefeably over 10) continues for at least 3 m below pile bottom may be considered as a suitable bearing stratum.

4.3 Review of Development Activities Proposed in the Low Lying Areas

The development activities proposed in the low lying areas in and around Colombo City include, light industries, warehousing, commercial and office buildings with main thrust of development concentrated on the housing sector. The types of structures that are to be constructed, and the estimated loads that will be transmitted to the ground are given in Table 1.

TABLE: 1 - TYPE AND LOADS OF STRUCTURES

Type of Structure	Lay out	Type of footing	Column/wall loads
1 RESIDENTIAL			
Single storeyed		Strip footing	25 KN/M
2-storeyed	Load bearing	-do-	65 KN/M
3-storeyed	wall spacing	-do-	115 KN/M
4-storeyed	= 3.0 m	-do-	165 KN/M
2-storeyed framed	3.5 x 3.5	pad footing	360 KN
3-storeyed framed	grid	-do-	580 KN
4-storeyed framed		-do-	820 KN
2 OFFICE			
3-storeyed framed	4.5 x 4.5	-do-	1380 KN
4-storeyed framed	grid	-do-	1900 KN
3 LIGHT INDUSTRIAL		-do-	65 KN
4 HEAVY INDUSTRIAL		-do-	175 KN

4.4 Fabrication of Under-reaming Equipment

The applicability, economy and performance of under-reamed piles installed in subsoil conditions prevailing in the low lying areas, largely depend on the configuration of under-reamed bulb, and the method of installation. Therefore, prior to installation of any prototype piles in the field, model tests were carried out to study the performance of installation equipment and technique. Based on the results of model tests conducted at AIT, the following equipment were fabricated by NBRO to enable installation of piles with nominal shaft diameter of 250 mm and under-reamed bulb of 625 mm diameter.

1) A Spiral Earth Auger (250 mm ϕ) to excavate the hole for pile shaft,
2) A Sand Bailer (125 mm ϕ) with two flap valves at the bottom,
3) A bucket type Slime Remover(200 mm ϕ) with flap valve for cleaning the borehole,
4) A Guide Casing (300 mm ϕ) to align the initial verticality of the borehole
5) An Under-reaming Tool capable of making a cavity for 625 mm ϕ) bulb,
6) A Conical Hopper and a set of Tremie Pipes(125 mm ϕ) for pouring concrete.

4.5 Installation of Trial Piles

As the first step, it was considered necessary to install several prototype piles in different soil conditions to examine construction problems in excavation, under-reaming and casting of concrete and to study variations of installation techniques with a view to improve the tools and techniques for increased efficiency.

Twelve sites in the different zones of low lying areas were selected for this purpose with due consideration given to;

1) Subsoil conditions,
2) Accessibility for installation equipment, material and crew,
3) Adequacy of space for construction activities,
4) Availability of the site free from interruptions for the duration of entire field operations.

A number of auger holes were advanced initially to obtain subsoil information speedily and economically within the selected sites, and Standard Penetration Tests were conducted to determine the strength profiles at locations identified for casting the piles. A total of 18 trial piles were installed in different subsoil conditions, and these were exhumed to examine the performance of the equipment and technique and to make necessary modifications and improvements.

4.6 Load Testing of Piles

After being satisfied with the performance of the trial piles several test piles were installed for load testing to determine ultimate capacity of the pile in (i) axial compression (ii) axial tension and (iii) lateral thrust. Load testing of piles with a bulb diameter of 625 mm supported on a loose to medium dense silty sand layer (SPT N-Value 7 - 11) indicated that an ultimate axial load of about 20 ton could be easily achieved. However, due to the weak nature of the soil surrounding the pile shaft, appreciable pulling resistance or lateral resistance could not be achieved.

Placing the bulb near the surface of a hard stratum may be reasonable when upward dragging forces or lateral forces are not likely to act on the pile and when the thickness of the bearing stratum is not very large. Although it may appear economical to cast the under-reamed bulb at an elevation as high as possible within the hard stratum, a major problem experienced is the collapsing or falling of the overlying weak soils into the bulb cavity. Therefore to ensure stability of the bulb cavity, adequate thickness of hard stratum should be allowed above it to effect arch-action. In the second series of testing, the bulbs were installed within the hard stratum leaving a minimum of 1 m depth below its interface with the weak layer. These piles are now being load tested.

5.0 DESIGN ASPECTS OF UNDER-REAMED PILES

5.1 Stability Criteria

Under-reamed pile foundations, like any other foundation, must fulfill two criteria for satisfactory performance. It should be structurally safe and it should not fail either in shear or by showing excessive settlement.

5.2 Safety of Pile as a Structural Member

The compressive load transmitted to the pile from the superstructure is first taken by the pile top and is distributed through skin friction and bearing before finally reaching the toe. Therefore, the pile should satisfy the design criterion as a short column in sustaining the design loads considering both concrete and steel in pile section. By ultimate load theory, the ultimate capacity of pile in compression is given by:

$$Pu = 0.4\ Ocu\ (Ac - As) + Osy\ As \quad - (1)$$

Where, Pu = ultimate capacity of pile in compression

Ocu = ultimate cube strength in the compression

Osy = yield strength of steel reinforcement

Ac = cross-sectional area of pile stem

As = cross sectional area of longitudinal steel.

The under-reamed bulb portion is not provided with any extra reinforcement. The shape and size of bulbs and transference of loads are such that the concrete section alone is sufficient.

5.3 Bearing Capacity from Soil Properties

The ultimate capacity of piles can be estimated based on soil parameters determined from field and laboratory tests. The ultimate bearing capacity Qu of a multi under-reamed pile can be obtained from the expressions in Annex 1.

These formulae suggested by CBRI (1978) need modifications in applying to under-reamed piles installed through weak organic soils in marshy areas. The skin friction component cannot be considered as a positive contribution to pile capacity since the surrounding weak soils settling under imposed loads will develop negative skin friction instead.

5.4 Safe Load from Pile Load Tests

A direct method for determining the safe load of a pile is to carry out a pile load test which may be used for design when done in advance, or as a check for piles already cast. However, pile load tests are costly and time consuming and may be permitted where a large number of piles are involved in a project. CBRI recommends a minimum of two pile load tests for a sizable work (more than 200 piles) where detailed information about the soil strata and past experience is not available. Therefore in order to adopt under-reamed piles as an effective low cost foundation type in Sri Lanka, it would be necessary to establish sound empirical formulae proven with load testing so that pile capacity could be easily estimated using soil parameters obtained through less costly field and laboratory investigations. Preliminary load tests carried out on prototype piles have indicated general agreement of the CBRI formule to local lateritic soils. However, this needs confirmation through further tests.

6.0 INSTALLATION OF UNDER-REAMED PILES THROUGH WEAK SUBSOILS

6.1 Installation Procedure

The procedure adopted by NBRO for the installation of prototype under-reamed piles for the research project is as follows.

The location suitable for installing the pile and the elevation of the under-reamed bulb are determined after borehole investigations. After setting the tripod in position, a hole is excavated at the intended pile location to install the guide casing for initial alignment of the hole. A boring guide is then set in position to ensure verticality of the hole at later stages of excavation. The hole for the pile stem is advanced using a spiral auger while maintaining the hole full with bentonite slurry to prevent collapsing of surrounding soil. Once the required depth is reached the bottom of the hole is cleaned using the slime remover. Cavity for the bulb is then under-reamed using the reaming tool. Soil deposited in the bucket attached to the reaming tool is removed from time to time as required. After ensuring under-reaming is carried out to the required size, the bottom of hole is cleaned once again and the boring guide is removed. Steel reinforcement cage is then placed in position and the tremie pipe is inserted to reach the bottom of hole. The lower end of the pipe is fitted with a mortar plug which prevents entry of bentonite in to the pipe, but can be easily pushed out under the pressure of concrete filled in the pipe (A modification to this is a valve provided at the neck of the hopper receiving concrete.). Concrete is then poured in with gradual lifting of the pipe while ensuring that its lower end is always kept within the concrete poured, so that mud does not intrude into the concrete.

6.2 Problems Associated with Installation through Weak Subsoils

Considering the weak nature of the superficial soil strata, piles are likely to be constructed only after the marshy land is reclaimed to offer suitable working space. Excavation of a large diameter borehole through a clayey filled layer or through lateritic ground at deeper elevation could be done without much difficulty. However, advancing a borehole through peat, soft clay or loose sandy or silty deposits below ground water table is extremely difficult without using bentonite slurry or casings for preventing the walls from collapsing. In a background of suggesting under-reamed piles as a low cost technique, use of casings is not practical since driving and withdrawal of casings become more difficult. Therefore, bentonite slurry of an optimum concentration should be used also considering the efficiency of tremie concreting.

During or after the under-reaming process, the column of weak subsoil above the bulb tends to cave in if a sufficiently thick layer of hard material, which can effect arch-action, is not available immediately above the bulb cavity. Therefore no time should be lost between under-reaming and concreting operations.

Another problem encountered is the bulging of the pile stem due to inadequate lateral support from the surrounding weak soils. Bulged sections with diameters as large as 350 mm were observed in some piles which were intended to be 250 mm - stem.

The reaming tool used in the early test piles were found to be rather heavy and not efficient in manual operations. A new lighter tool was therefore designed and this is now being tested.

7.0 IMPACT ON THE INDUSTRY

7.1 Applicable Areas

The under-reamed piling technique is a simple low cost and manually operable piling method which does not require sophisticated tools and equipment. The technique when developed to suit the local subsoil conditions is expected to have a great potential in Sri Lanka, as an economical foundation types applicable to structures with moderately loaded foundations. Applicability of the under-reamed piles will not be limited to the construction in reclaimed marshy lands but also to highlands.

In the Colombo region, specially where a high demand for low-rise buildings will exist in the near future, it is observed that even in most highlands the uppermost layers of the ground are generally weak and the foundations are required to be placed at deeper elevations by-passing the weak layers. Deep excavations sometimes become very difficult due to high ground water table and other environmental conditions, such as, presence of adjacent buildings etc., which could be affected. In such situations, under-reamed piles appear to be a suitable solution.

Under-reamed piles will have potential use in some areas of the the dry zone where expansive soils exist, in controlling distress to lightly loaded structures (caused by foundation movements due to swelling and shrinking associated with moisture variations.

The under-reamed piles would be also effective in resisting uplift forces in tall structures affected by wind, cable tension etc., as in towers and pylons, and also in retaining walls and bridge abutments, where lateral and uplift forces are significant.

7.2 Cost Aspects

During the research and development stages of the under-reamed piling techniques suitable to low-lying areas, the average cost of installation of a 5 m long pile with 250 mm - stem and 600 mm - bulb was found to be in the order of Rs.2000 at 1988 prices). Costs have been worked out for the items excluding mobilisation and demobilisation, costs of which should be distributed over a large number of piles for a particular site. With further improvements to the techniques, operation and management the cost could be reduced substantially on commercial basis.

8.0 REFERENCES

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Senanayake (1986) "Geotechnical Mapping of Low-Lying Areas in and around Colombo City". Asian Regional Symposium on Geotechnical Problems and Practices in Foundation Engineering, Colombo 1986.

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BEARING CAPACITY FORMULAE FOR UNDER-REAMED PILES

For sandy soils the following expression may be used:

$$Q_u = A_p.(\tfrac{1}{2}.D.\gamma.N_\gamma + \gamma.d_f.N_q) + A_a\,[\tfrac{1}{2}.D_u.n.\gamma.N_\gamma + \gamma.N_q.\sum_{r=1}^{r=n} d_r] + \tfrac{1}{2}.\pi.D.\gamma.K\tan\delta \times (d_1^2 + d_f^2 - d_n^2)$$

For clayey soils the ultimate load carrying capacity of an underreamed pile may be worked out from the following expression—

$$Q_u = A_p.N_c.C_p + A_a.N_c.C_a' + C_a'.A_s' + \alpha.C_a.A_s$$

where

Q_u (kg) = ultimate bearing capacity of pile

A_p (cm²) = $\pi/4.D^2$, where D (cm) is stem diameter

A_a (cm²) = $\pi/4(D_u^2 - D^2)$ where D_u (cm) is the underreamed bulb diameter

A_s (cm²) = surface area of the stem

A'_s = surface area of the cylinder circumscribing the underreamed bulbs

n = number of underreamed bulbs

γ (kg/cm³) = average unit weight of soil (submerged unit weight in strata below water table)

N_γ and N_q = bearing capacity factors depending upon the angle of internal friction φ, given in Table A.

N_c = bearing capacity factor, usually taken as 9

K = earth pressure coefficient (usually taken 1.75 for sandy soils)

δ = angle of wall friction (may be taken equal to the angle of internal friction φ)

d_1 (cm) = depth of the centre of first underreamed bulb

d_n (cm) = depth of the centre of the last underreamed bulb

d_r (cm) = depth of the centre of different underreamed bulbs below ground level

d_f (cm) = total depth of pile below ground level

C_p (kg/cm²) = cohesion of the soil around toe

C'_a (kg/cm²) = average cohesion of soil around the underreamed bulbs

α = reduction factor (usually taken 0.5 for clays)

C_a (kg/cm²) = average cohesion of the soil along the pile stem

Table A

Bearing Capacity Factors

φ	N_γ	N_q*
20	3	10
22	5	12
24	7	15
26	9	18
28	13	22
30	18	29
32	25	37
34	34	50
36	47	65
38	65	90
40	85	128

*To be reduced to half for bored piles including underreamed piles.