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Braced Excavation in Soft Clay – Experiences of Calcutta Metro Constructuion

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Introduction

Although I have spent the best part of my professional career in academics I had been fortunate enough to be associated with many field problems involving construction of foundations and substructures in weak soils, in particular soft clay. Quite apart from the fact that such problems require understanding of the fundamental nature of soft clay as engineering material they often give rise to unusual problems of stability and deformation which need practical and non-conventional approach to their solution. In many such problems, field measurements and instrumentation help not only to provide insight into the relevant soil–structure interaction phenomenon but to develop rational methods of analysis and design and identify construction control parameters. Of the many such problems I have been associated with during the last twenty-five years, none could match the challenges that were posed by the construction of Calcutta Metro. I got involved in the project sometime after its inception and my continued association with the project for almost fifteen years since then kept me abreast with all the ups and downs, technical or otherwise, the project had to go through till the construction was over in 1995. Today I take the opportunity to share with you my experiences with the design, construction and performance of braced cuts as adopted for the 16.63 km long phase I of Calcutta Metro passing mostly through the soft alluvial soil of Normal Calcutta Deposit below one of the most heavily built up metropolitan cities of India.

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Calcutta Metro in Perspective

The city of Calcutta lies in Gangetic West Bengal - about 200 km north of the Bay of Bengal, Fig.1. Calcutta Metropolitan District (CMD) with an area of 1250 sq. km and a population of more than ten million has grown along the banks of the river Hooghly with the river running along the western boundary and separating it from the neighbouring city of Howrah, Fig.2. Calcutta is the major centre of industrial and commercial activity in Eastern India and has grown in population rapidly since the second world war. The growth increased many folds after the partition of India in 1947 due to a large influx of people from erstwhile East Pakistan. The phenomenal growth in population put pressure on the civic services including transport. The meagre network of roads made the situation chaotic.

The inherent problems of transport in Calcutta arise out of the fact that roads in the CMD account for only 6.2% of the urban land which is much below the national average, not to speak of the 30% requirement of modern cities. The city is surrounded by the railway tracks of Eastern Railway and Calcutta Port, but these have not been used much for urban transport except in recent years when the Circular Railway was partially introduced, Fig.3.

The need for a rapid transit system for Calcutta had been felt for along time. After numerous studies by national and international committees a master plan for Rapid Transit System (RTS) aggregating a total length of 97.5 km was prepared, Fig. 4. The construction of the first RTS line or the Metro covering a length of 16.633 km from Dum Dum to Tollygunj was commenced in 1974.

Geological Features of Calcutta Area

The Calcutta Metropolitan District lies in the deltaic plains of South Bengal. The river Hooghly runs North South along western flanks of the city and is linked to Howrah by four bridges in its 45 km stretch from Princep Ghat to Naihati.

The Calcutta soil forms part of the Bengal basin, which is believed to have its geological history dating back to the middle of the Mesozoic era. The basin was the site of continuous deposition from the Cretaceous period. In the Tertiary period the sediments on the Mesozoic floor were mostly deposited by the Ajay and Damodar river systems on the West and from the Assam Plateau on the east. Subsequent Pleistocene sediments consisting of succession of gravel, silt and sand are believed to have been deposited on the tertiary beds (Coulson, 1940).

The subsoil in the upper strata of the Bengal basin, which is of

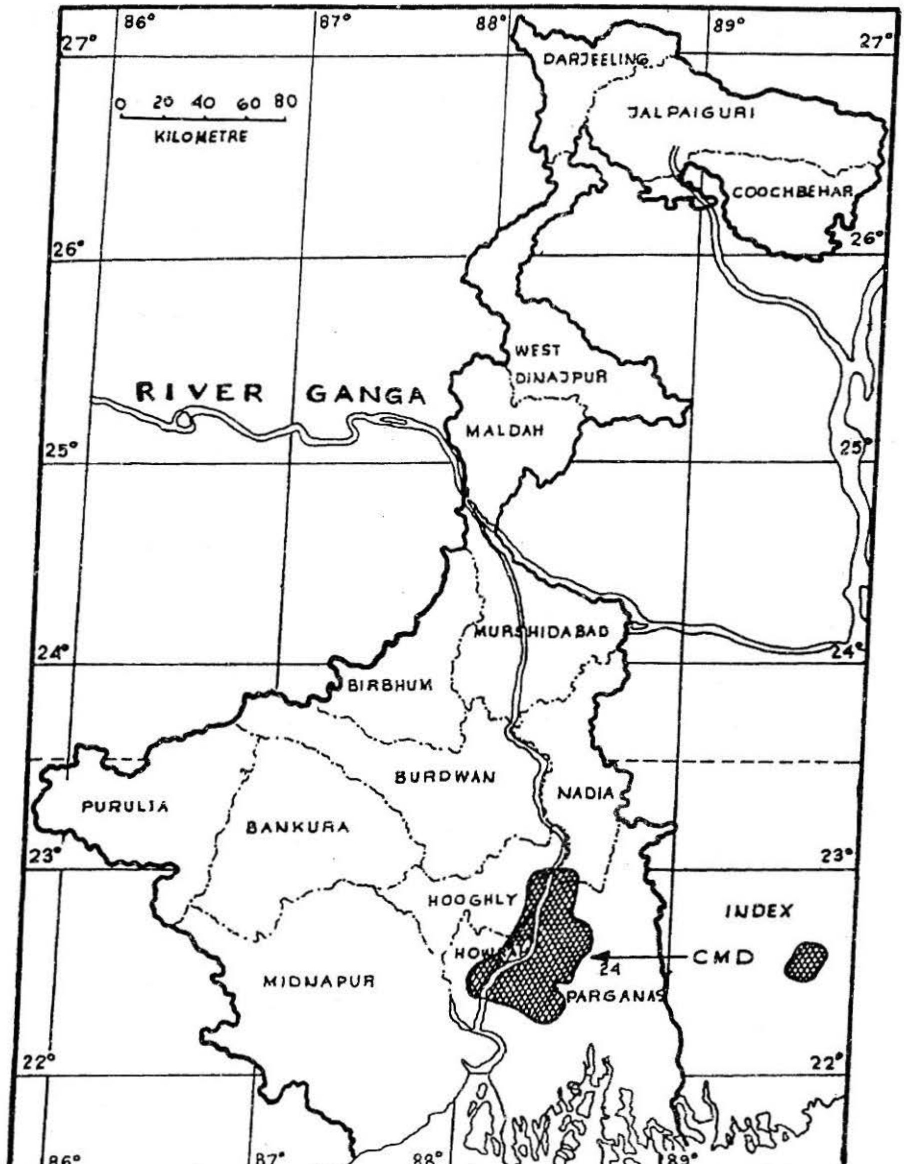

FIGURE 1 : Map of West Bengal showing Location of Calcutta Metropolitan District

FIGURE 2 : Calcutta Metropolitan District

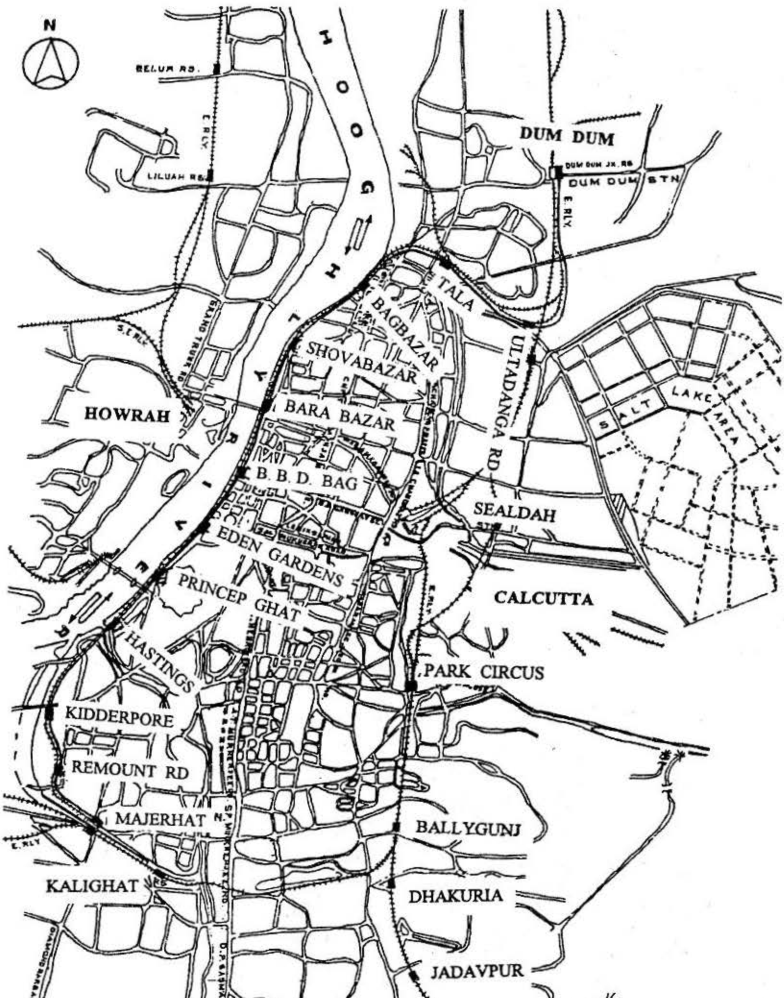

FIGURE 3 : Circular Railway for Calcutta

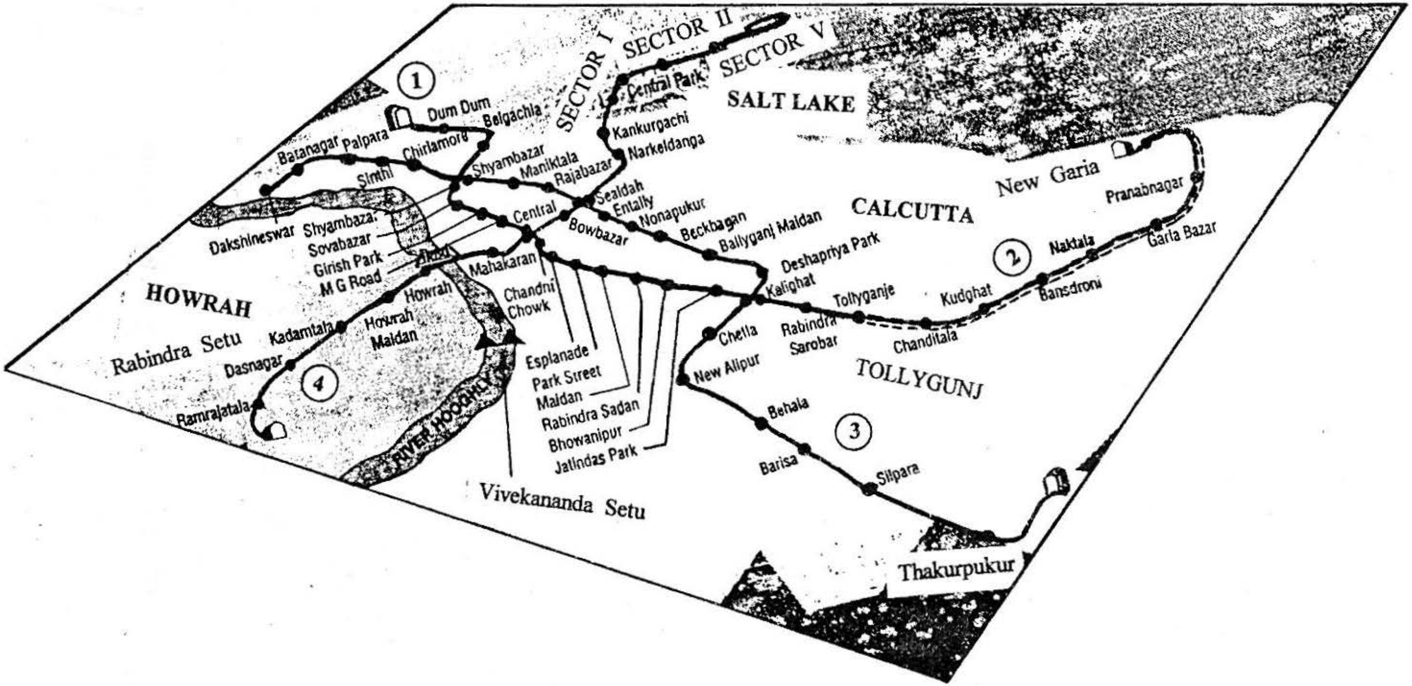

FIGURE 4 : Proposed Rapid Transit System for Calcutta

immediate relevance to foundation engineers, is, however, of recent origin and is believed to have been deposited by the Ganga river system. Opinion differs as to boundary between the Pleistocene and recent deposits in the area, but there seems little doubt that the top 100 metres of the sediments, at least, are of recent origin. Deposition has taken place under typical alluvial environment - in the form of back-swamp deposits, meander belt deposits and channel fill deposits (Dastidar, 1967; Ghosh and Gupta, 1976; Som, 1985).

The terrain of Calcutta is almost flat with a general elevation of about 5m above MSL. The ground is higher near the river banks but slopes down towards the low-lying marshy lands in the east. The low-lying areas of the city often get flooded by the monsoon rains.

System Design and Metro Alignment

The building of a metro railway involves construction of a subway box between two ends of the alignment with platforms and passenger dispersal facilities at stations at regular intervals. Most parts of Calcutta Metro have been designed to remain underground except near the two terminal points where they end in car sheds on the ground. The Calcutta Metro alignment (16.633 km) is made up the following sections,

Elevated	:	1.228 km
Surface	:	0.270 km
Underground	:	15.135 km

The underground stretch of 15.135 Km was built by a number of methods:

Open cut	:	0.309 km
Braced cut:		
Sheet piles	:	0.365 km
H-pile with Timber Lagging	:	0.684 km
Diaphragm wall	:	12.687 km
Shield Tunnelling	:	1.090 km

Figure 5 shows the location of stations on the Metro alignment along with the sections built by different methods. Cut and cover construction was mostly adopted below the major North - South consider of the City, viz.,

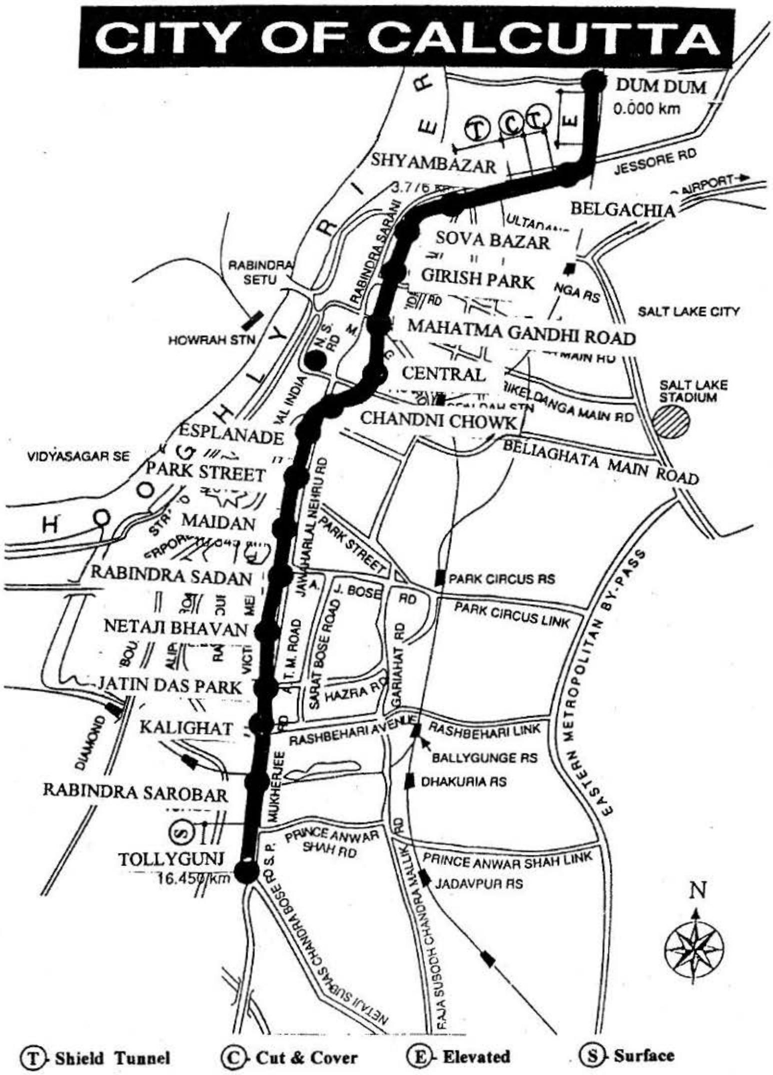

FIGURE 5 : Calcutta Metro Alignment

Chittaranjan Avenue, J.L.Nehuru Road, Asutosh Mukherjee Road and Shyamaprasad Mukherjee Road. Shield tunnelling was resorted to between Shyambazar and Belgachia to facilitate the alignment going underneath a railway yard and a busy Circular Canal.

General Subsoil Condition

Extensive soil exploration was done in the early seventies for the metro construction in Calcutta. Boring was done along the entire route for mapping the soil data. Preliminary investigation was done at intervals of 500 m. Later detailed soil exploration was done with 30 m deep boreholes at 100 m intervals. From the boreholes undisturbed samples were collected for laboratory tests. Also extensive field tests (e.g., standard penetration tests, dynamic cone tests, field permeability tests etc.) were done. The subsoil data along the metro alignment generally indicate weak cohesive strata in the upper layers except for a thin surface crust which appears firm by desiccation. The soil consists generally of silty clay / clayey silt, particularly soft in the top 12-14 m, having high organic content and decayed vegetation. The consistency of the soil improves in the deeper strata and alternate layers of firm to stiff silty clay are found below 14 m depth. Early geological explorations suggest that these alluvial deposits extend to several thousand metres below ground level. No borehole has yet been done to pierce the alluvium and reach the bed rock. However, records of 300 m deep boreholes reveal the existence of numerous layers of peat. kankar and clay beds interspersed with water bearing sand strata. Some remains of fossils have been found below 250 m depth (Coulson, op cit).

For the purpose of metro construction the soil in the top 30 m is of immediate relevance. In this zone, the soil consists mostly of silty clay/ clayey silt strata with laminations and occasional lenses of silt and fine sand below 20-25 m depth. This is characterised as Normal Calcutta Deposit, Fig 6. This general pattern is, however, broken at three locations, viz., Tollygunj, Maidan and Belgachia, by a somewhat different depositional feature. This second group of deposits is called the River Channel Deposit of the Adi Ganga and consists of fine sand going down to some depth, but ultimately resting on the deeper clay beds of the Normal Calcutta Deposit at 16-20 m depth, Fig.7. The deeper sand of the Normal Calcutta soil is golden brown in colour and has different grain size distribution as compared to the river channel deposits found at locations mentioned above. Figs. 8.1-8.2 show the longitudinal profile of soil strata for the entire metro alignment from Dum Dum to Tollygunj. The subway box passes mostly through the grey/dark grey silty clay/clayey silt of stratum II. Fig.9 shows a comparison of the soil profile as revealed by the excavation at Km 12 with that obtained from the soil exploration.

DEPTH (m)	Description of Strata	Soil Properties (average) N blows/30 cm	w (%)	LL (%)	PL (%)	C_u (t/m^2)	m_v (m^2/t)
0	I Light brown/brownish grey silty clay/clayey silt with occasional lenses of fine sand	5-8	30	54	24	4.0	0.002
4	II Grey / dark grey silty clay, clayey silt with semi-decomposed timber pieces	2-4	55	12	28	2.5	0.007
4	III Bluish grey silty clay with calcareous modules	10	30	60	25	6.0	0.002
20	IV Brown/yellowish brown sandy silt/silty fine sand with occasional lenses of brown and grey silty clay	20	38	20	30	4.5	--
25	V Mottled brown / grey silty clay with laminations often with rusty brown spots	20–30	25	65	25	10.0	0.001
30	VI Brown / light brown silty fine to medium sand	40	$\phi' = 40^\circ$				

FIGURE 6 : Subsoil of Calcutta : Normal Calcutta Deposit

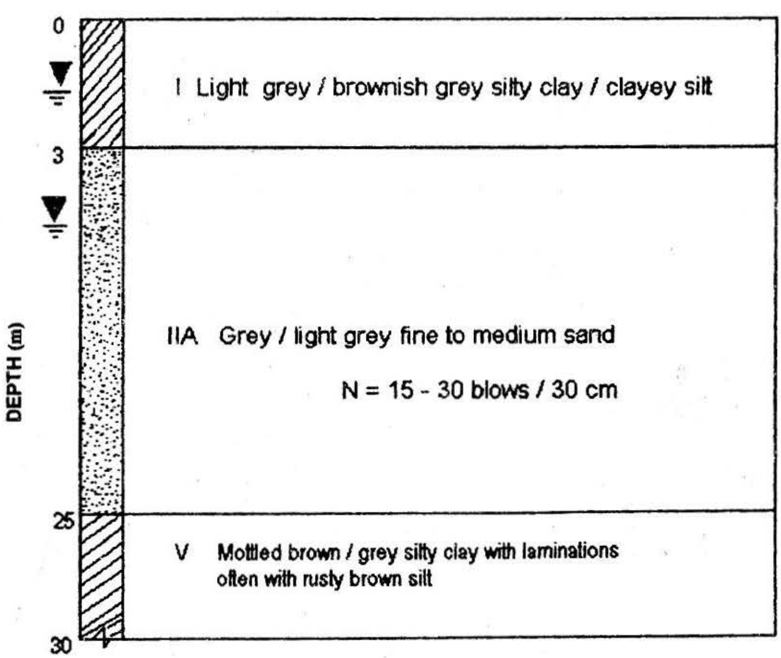

FIGURE 7 : Subsoil of Calcutta : River Channel Deposit

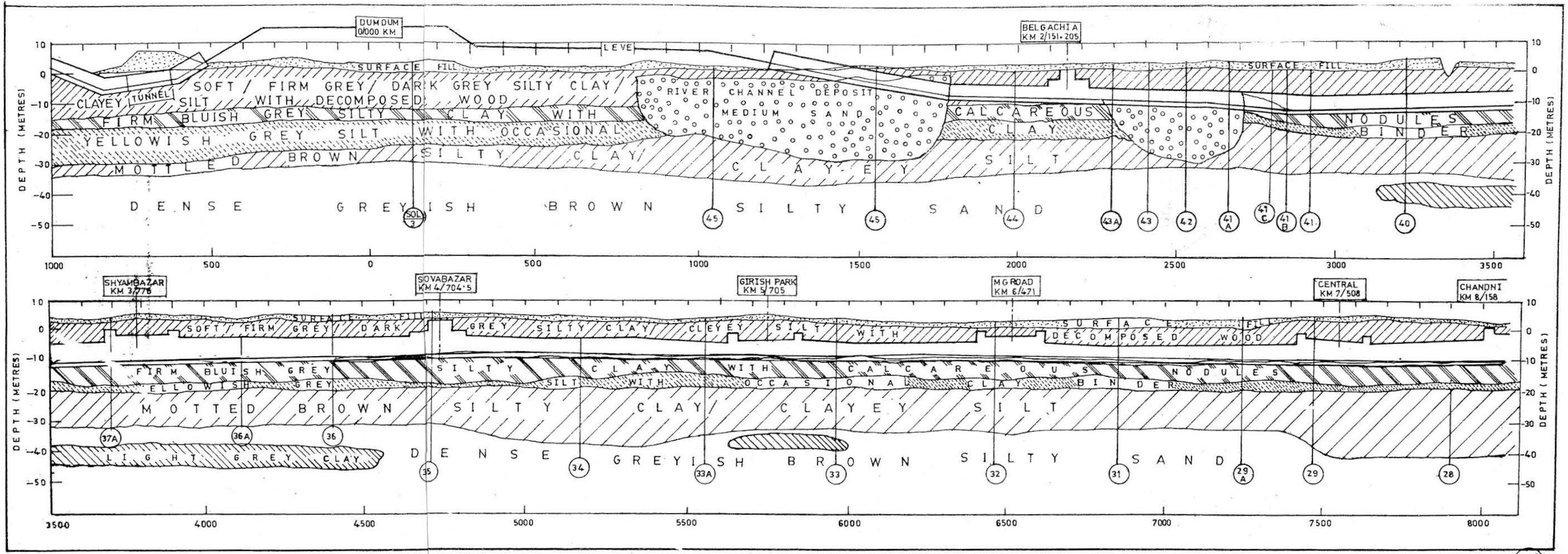

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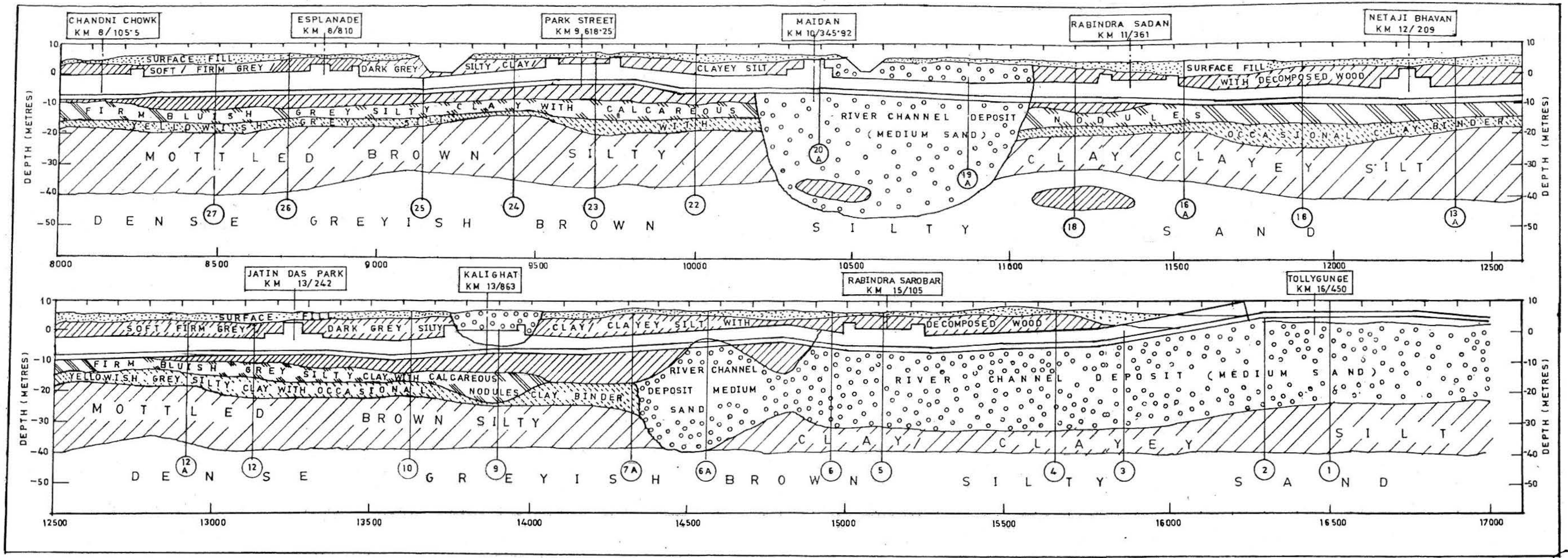
FIGURE 8.1 : Subsoil Profile of Calcutta Along Metro Alignment



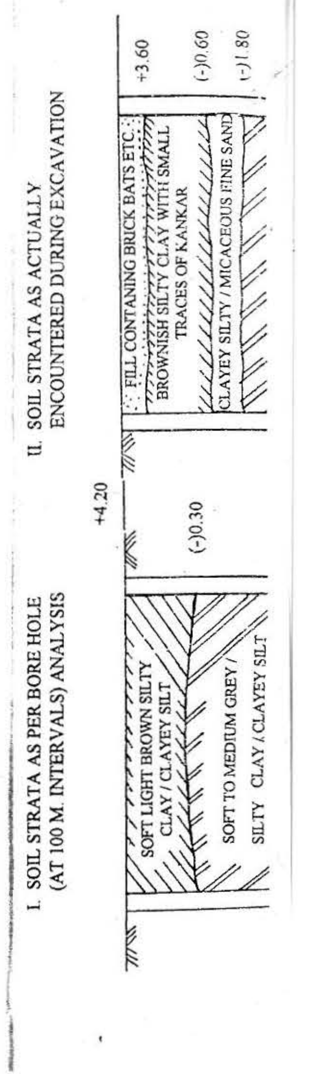


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FIGURE 8.2 : Subsoil Profile of Calcutta Along Metro Alignment

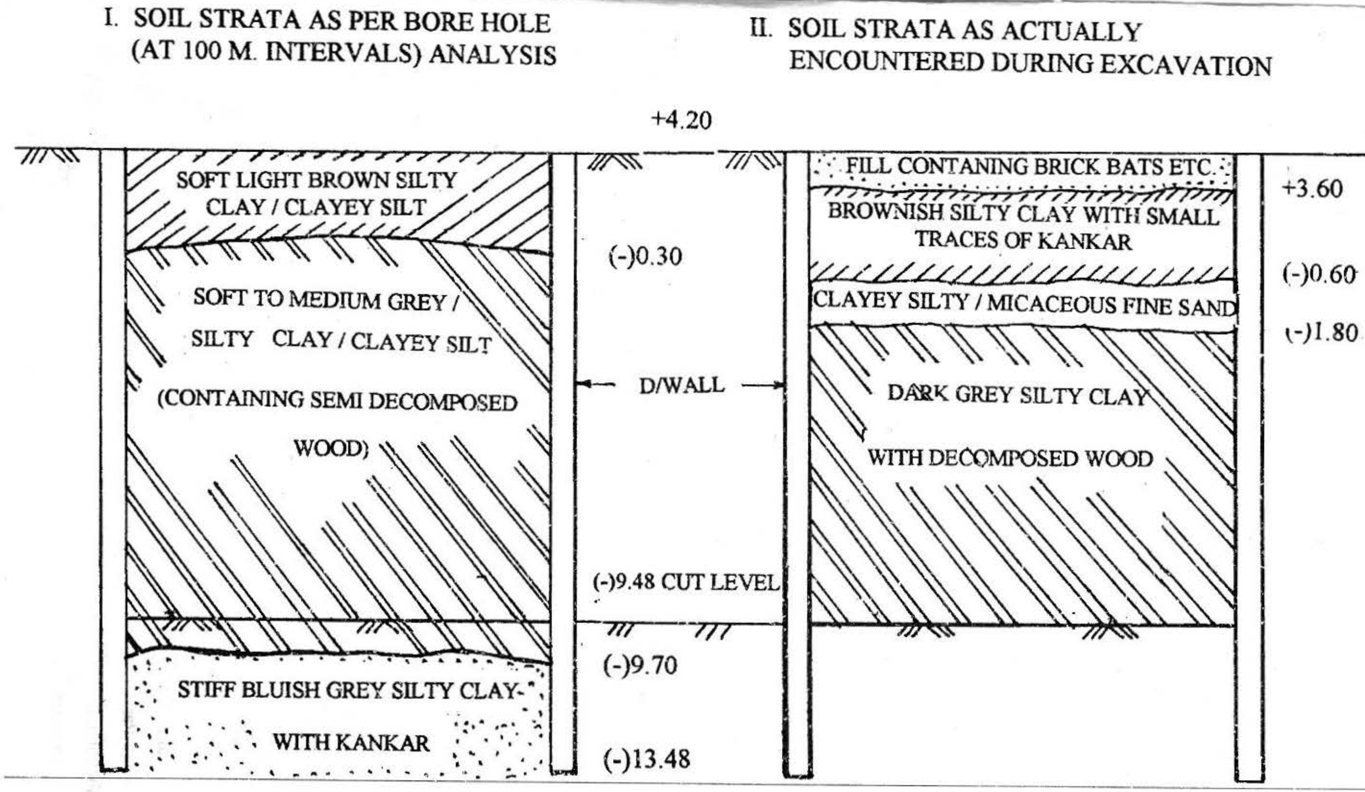


FIGURE 9 : Soil Strata Obtained from Soil Exploration and Field Excavation

Normal Calcutta Deposit

The general characteristics of Normal Calcutta Deposit, including the properties of individual strata are shown in Fig. 6. The upper region of Normal Calcutta Deposit consists of desiccated brownish grey/light brown silty clay followed by dark grey silty clay with decayed vegetation. These two strata constitute the softer components of the Normal Calcutta deposit and extend to approximately 14 m below G.L. Stratum II, in particular, has high water content and low undrained shear strength (25 kN/m^2). The deeper strata are stiffer in comparison and comprise of alternate layers of stiff to very stiff blue/ mottled brown silty clay. These are followed by dense brown sandy silt/silty sand down to 30 m below G.L. with 'N' value greater than 40.

River Channel Deposit

A deep deposit of grey silty sand with some clay and silt intrusions is encountered at three distinct regions, viz. Belgachia, Maidan and Tollygunj. This appears to have cut through the Normal Calcutta deposit to a depth of about 20 m below G.L.. At some locations, the upper soil often exhibits the

Table 1
Road Level and Highest Flood Level at Station Locations

Name of Station	Level of crown of road (Metres)	Highest flood level (Metres)
Tollygunge	+4.98	+5.16
Rabindra Sarobar	+4.84	+4.87
Kalighat	+4.26	+4.36
Jatin Das Park	+4.58	+4.64
Bhowanipur	+4.34	+4.42
Rabindra Sadan	+4.17	+4.65
Maidan	+4.59	+4.63
Park Street	+5.10	+5.10
Esplanade	+5.32	+5.55
Chandni Chowk	+5.70	+5.80
Central	+5.06	+5.46
Mahatma Gandhi Road	+3.96	+4.64
Girish Park	+4.78	+5.41
Shovabazar	+5.44	+5.59
Shyambazar	+5.43	+5.53
Belgachhia	+4.00	+4.20
Dum Dum Jn.	Elevated station	

characteristics of the dying stages of a river channel where finer particles of clay and silt are mostly deposited. The river channel deposit, however, comprises mostly of silty fine to medium sand of loose to medium compaction.

Ground Water Table

The ground water table in the CMD is fairly high. There is a perched water table in the upper cohesive strata which undergoes seasonal fluctuation and rises almost to the ground surface during the monsoon. The piezometric head of water, is, however, found 6 m below ground level in the north to 9 metres below G.L. in the central and southern districts. This has been attributed to large scale pumping of water from deep tubewells for domestic water supply over a long period of time (Som 1986). Table 1 gives the high flood level at different station locations. In general the low permeability of the silty clay strata did not require any dewatering for the 12-14 m deep excavations except for areas with local pockets of sand where local pumping had to be resorted to. The river channel deposit, however, required well-point dewatering.

Methods of Construction

Open Excavation

The metro construction was commenced in 1974 with the open excavation for the subway boxes in the 300 m stretch between Park Street and Maidan. The alignment here passed through an open tank where open excavation was possible. The tank was dewatered by pumping and the slush at the bottom was removed. The final excavation was required down to 8 m below the tank bed. This was done with sloping sides, Fig. 10. The eastern

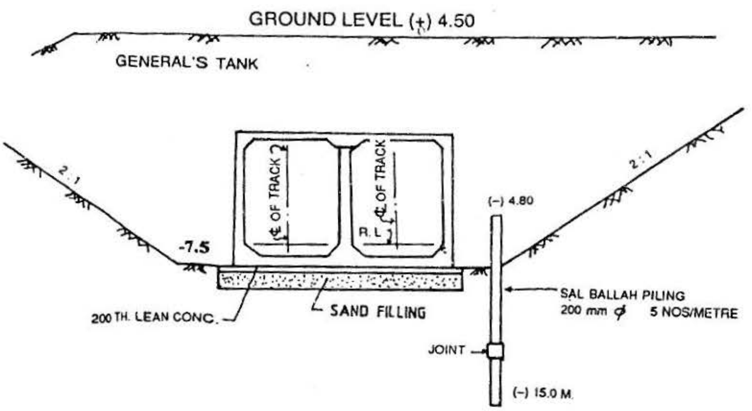


FIGURE 10 : Open Cut in Maidan Area

side of the sloped excavation with the tram track in close vicinity was protected by two rows of spliced timber piles. The excavation was done in alternate stretches of 3 m each and the elements of RCC box - each 21 m long - were built. Backfilling around and over the box was done by borrowed earth taking care to avoid differential filling on the two sides of the box. This was done to ensure that any unbalanced filling did not push the box out of alignment on any one side.

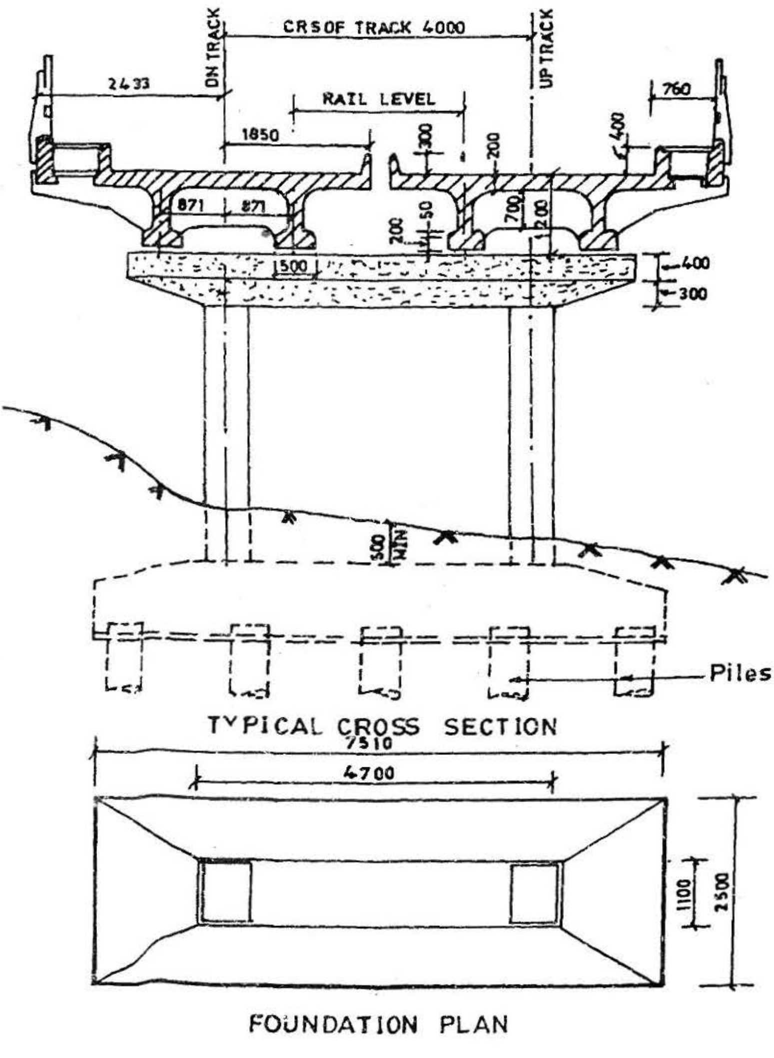


FIGURE 11 : Elevated Construction Near Dum Dum Station

Elevated Construction

Of the total length of the metro alignment 1.228 km near Dum Dum station was built on elevated structures. The running sections had independent decks for the two tracks but the station and approaches were provided on a single deck. The columns of the elevated structure were supported on R.C.C. pile groups at 4 m c/c with a 7.51 m long × 2.5 m wide pile cap supporting the twin columns of each track, Fig. 11.

Cut and Cover Construction

Most of the subway box for the Calcutta Metro has been built by cut-and-cover construction. The method essentially consists of putting two rows of vertical walls in the soil and suitably propping them against each other by steel struts as the excavation is made. When the final excavation level is reached the subway box is cast and backfilling done to restore the original ground surface as the struts are progressively removed. The walls are normally 10 m apart in the running sections and 19 m at station sections where the cut is made wider to accommodate the platform and other services. The depth of excavation varied from 12 m to 15 m. A typical excavation profile is shown in, Fig. 12.

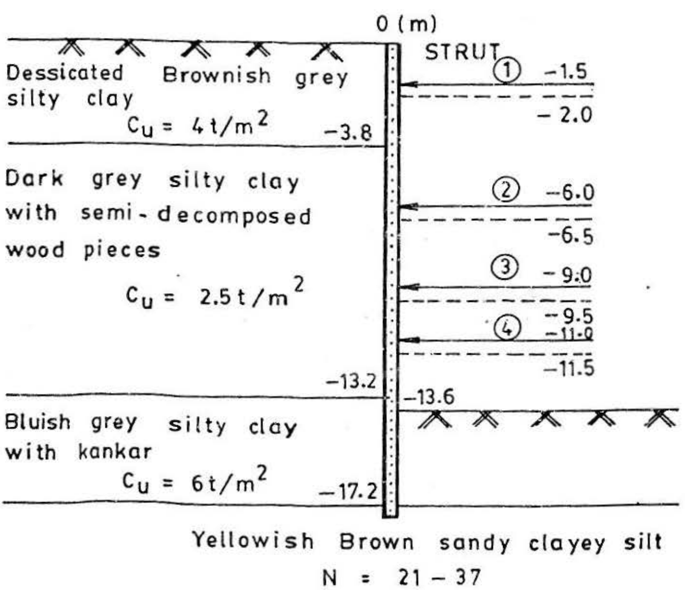


FIGURE 12 : Typical Excavation Profile for Cut-and-Cover Construction with Diaphragm Wall and Struts

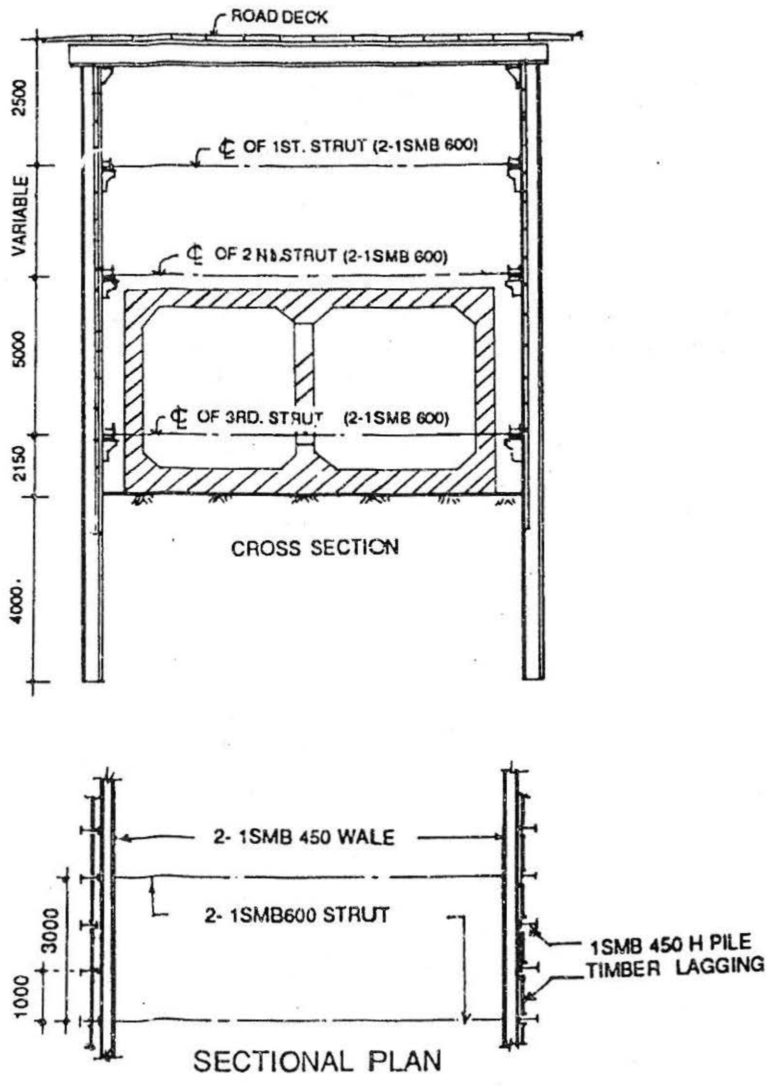
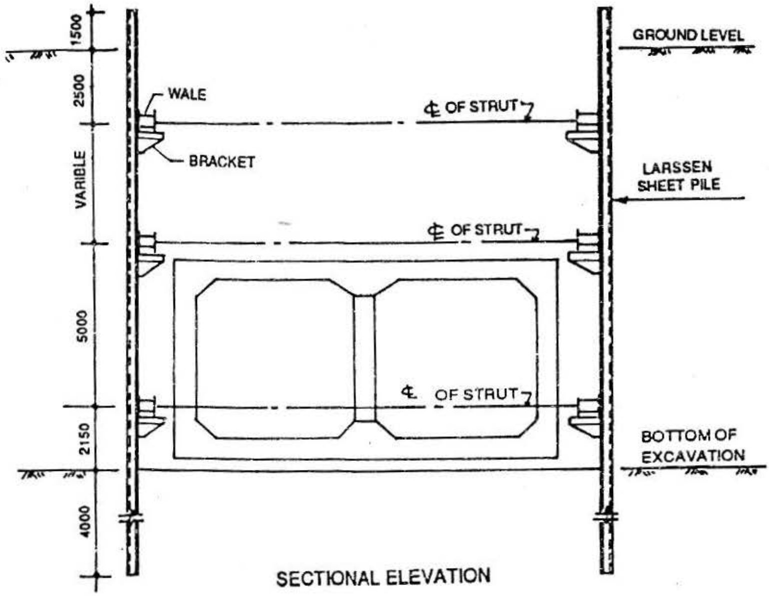


FIGURE 13 : Excavation with H-Pile and Timber Lagging

The retaining walls on two sides of the cut were made by steel sheet piles, H-piles with timber laggings or diaphragm walls. The original plan was to do the construction by sheet piles and diaphragm wall almost in equal lengths apart from some sections by H-pile with timber laggings. After some initial trials, H-pile with timber lagging was adopted only in the shallow depth of cut at the two ends of the alignment where the same rises up to the ground surface, Fig. 13. This method of construction gave rise to major soil



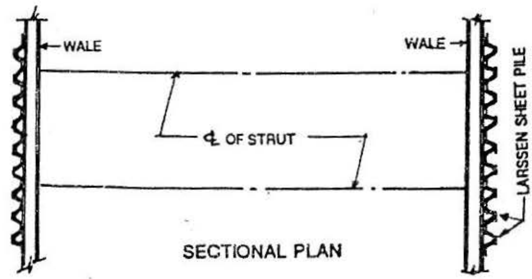


FIGURE 14 : Excavation with Sheet Pile and Struts

loss through the timber laggings and presented problems with dewatering in the absence of a deep cut-off. Steel sheet piles with struts had been used over a length adjacent to Belgachia station quite successfully, Fig. 14. But heavy sheet piles of the right type were not available indigenously. They had to be imported from Russia. This took time and attempts were made to use Indian sheet piles of thinner section modulus by providing more struts. Still, when full depth of cut approached, the clutches of the sheet pile started opening up under increasing earth pressure. This caused bulging of the sheet pile and consequent soil loss leading to large ground movement. This was not desirable on city roads with old buildings in close vicinity of the cuts. Also

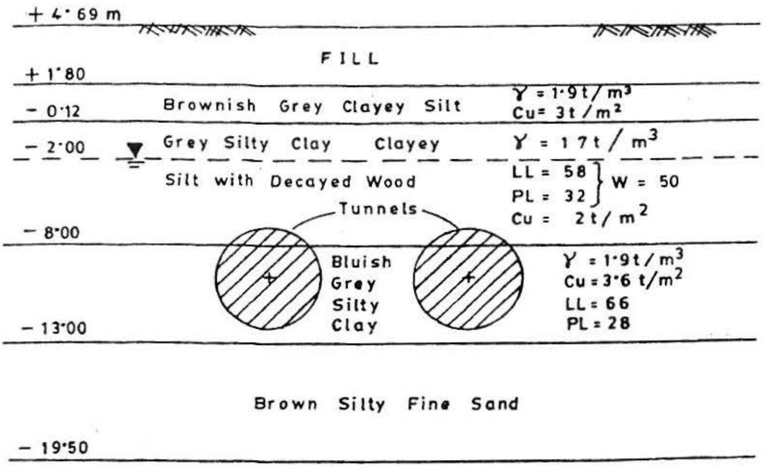


FIGURE 15 : Shield Tunnelling between Belgachia and Shyambazar

driving sheet piles or H-piles close to old buildings appeared hazardous and extensive road decking that would be needed to maintain the minimum traffic flow would not be possible on sheet piles. Learning from experiences of the early construction with sheet pile and H-pile. It was decided that the entire construction within the city area would be done by diaphragm walls and struts.

Shield Tunnelling

Shield tunnelling was done in two stretches between Belgachia and Shyambazar covering a total length of 1.090 km. The alignment had to pass below a heavily built-up area and a busy circular canal where Cut and Cover construction was not feasible, Fig. 15. A pair of tunnels, each 5.1 m diameter and spaced 11 mm c/c with their crown depth 10-11 m below G.L. were built by the shield tunnelling technique. Work was done under compressed air to improve the stability of construction and to reduce ground movement (Som and Narayan 1985, Som 1987,1997).

Braced Cut with Diaphragm Wall

Diaphragm Wall

Diaphragm walls were made by first building guide wall in the shape of "inverted L", for guidance of grabs during trenching, Fig. 16. They were made of M15 grade concrete about 100-150 mm thick. A gap of 650 mm was kept between the two guide walls for making 600 mm

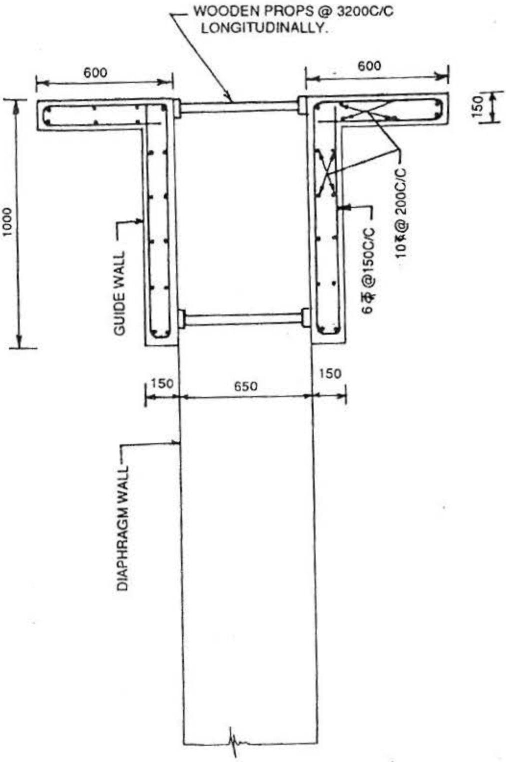


FIGURE 16 : Guide Wall for Diaphragm Wall Trench

diaphragm walls. The excavation was done in the trench between the two guide walls by Crawler mounted Kelly grab with a telescopic guide. The diaphragm walls were built in panels of 3 m normally and the depth varied from 16 m to 23 m. The sides of the diaphragm wall trench were stabilized by bentonite slurry of 1.07 to 1.10 specific gravity. The excavation time for each panel was 6 to 8 hours. After the trench excavation the reinforcement cage of the diaphragm wall weighing about 4 to 5 tonnes, Fig. 17, was lowered by a crane and concreting was done by tremie method with M20 concrete to replace the bentonite slurry which was pumped back to vats on the ground for re-cycling and reuse. The concreting was done continuously and took 3-8 hours for each panel. The cylindrical form tubes at the end of each panel ended in semi-circular concave shape and the space between two such panels was filled with panels of equal length having semi-circular convex ends. Thus, alternate panels of concave and convex ends formed an interlocked system of diaphragm wall.

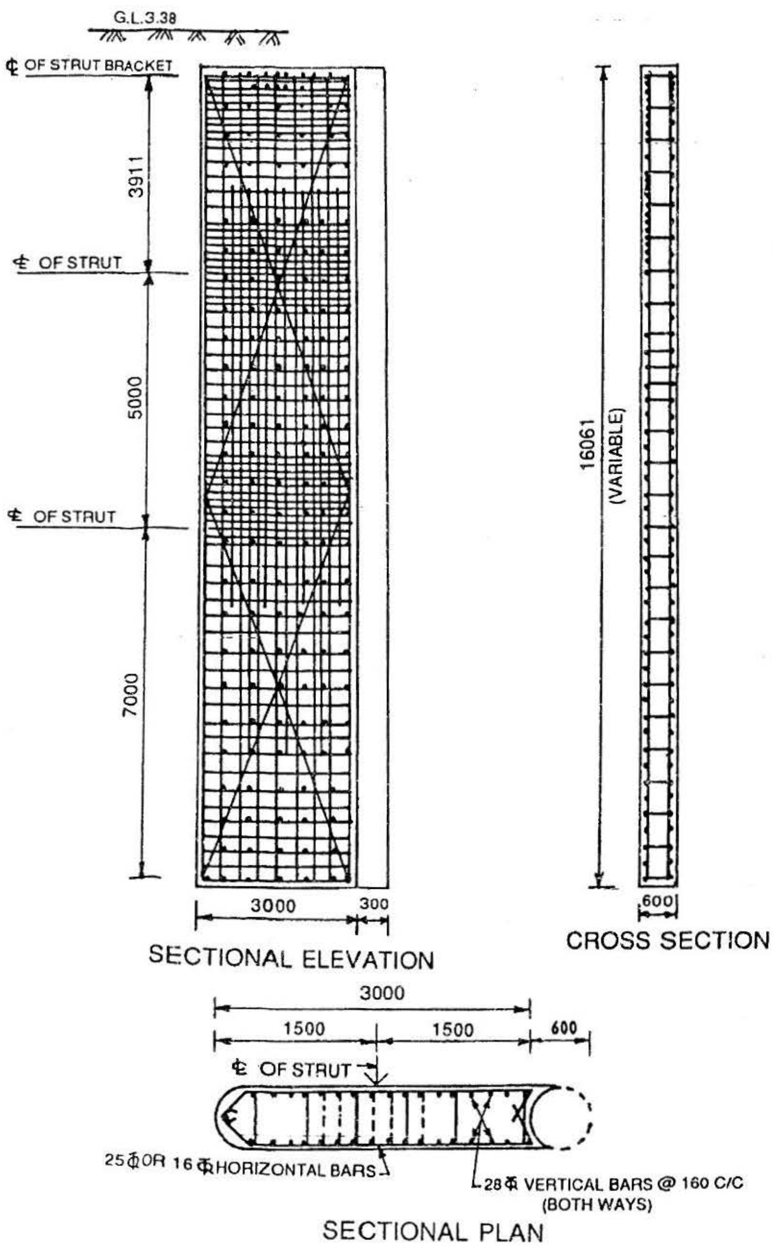


FIGURE 17 : Reinforcement Cage of Diaphragm Wall

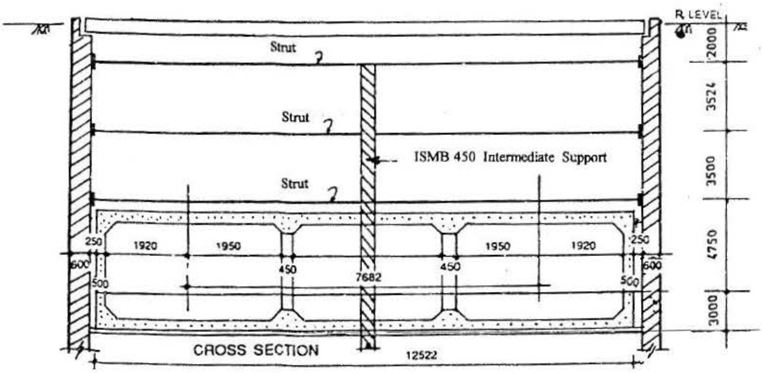


FIGURE 18 : Excavation for Wide Cuts with Intermediate Support

Struts

During excavation the diaphragm walls were held in position by multiple steel struts. In most case three or four struts were found adequate for the 12-14 m deep cuts, Fig. 12. In certain areas, e.g., Chitpur yard, where the depth of cut was 16 m, five layers of struts had to be used. The struts were placed generally @ 3 m intervals along the length. They were made of ISMB 400, 500 or 600 sections. As per design the top struts were lighter than the others. Accordingly, the latter were provided with heavier sections.

In the running stretches, the width of cut was 10 m and the struts were designed as horizontal compression members resting on brackets at two ends. For wider cuts an intermediate support was provided with a central vertical joist driven into the ground to reduce the effective length of struts. The struts were preloaded by hydraulic jacks immediately after installation so that they were held tightly in position against the diaphragm walls. Excavation was mostly done manually. Later, mechanical grabs were used in the northern stretches. Under the busy road, manual excavation could only be adopted.

Box Construction and Backfilling

After the bottom of excavation was reached a layer of mudmat, 300 mm thick, of M10 concrete was placed and the same was rammed and levelled. The bottom raft was then cast. This was followed by casting of side walls in convenient lifts and the struts were removed in appropriate sequence. In the latter stages of construction the side walls were cast through the struts which were subsequently cut away from inside the box. Attempts were made to use the diaphragm wall as the side wall of the box by casting the base raft integrally with the diaphragm wall, Fig. 16, but the idea had to be

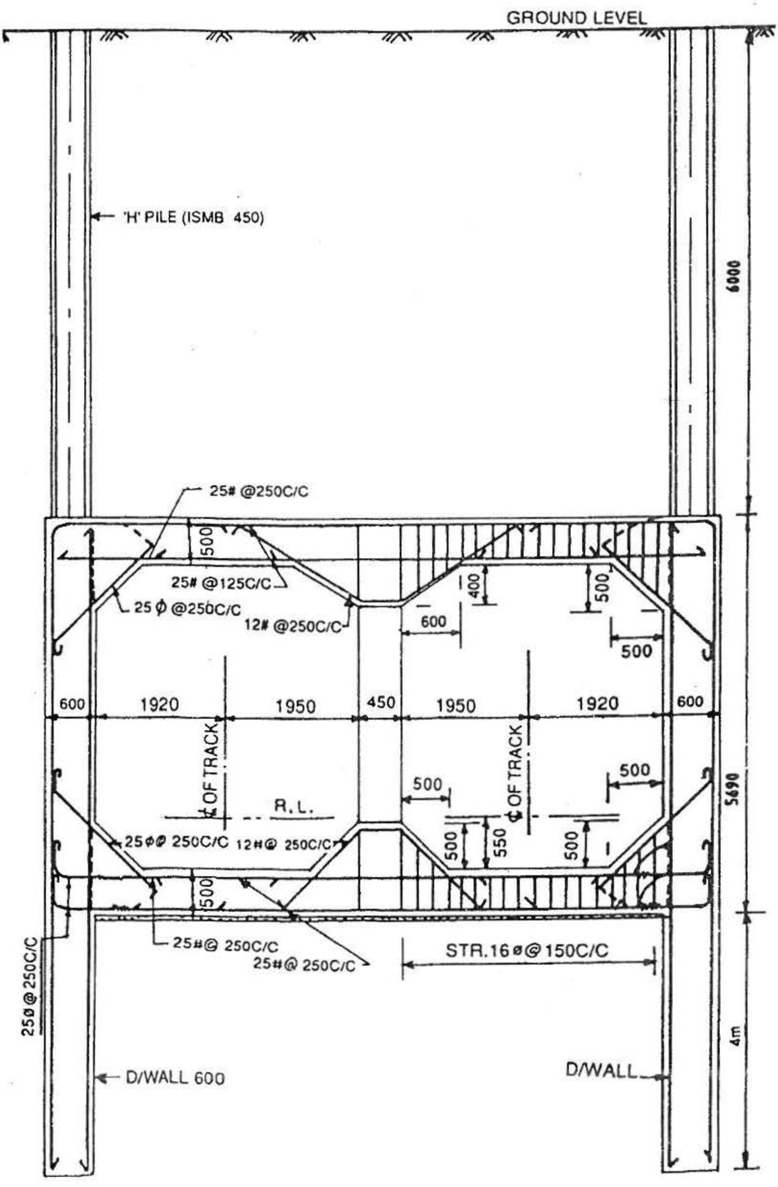


FIGURE 19 : Proposed Construction with Diaphragm Wall as Side-Wall of Subway Box

discarded because of lack of continuity of diaphragm wall and problems of leakage through the joints.

Design of Braced Cuts

The design of braced excavation involves two distinct, yet inter-related features :

a. Stability of the excavation, ground movement, control of water into the excavation, effect on adjoining structures, etc.

b. Design of structural elements, i.e., diaphragm wall or sheet pile, struts or anchors etc.

Although the overall stability of braced cuts in soft ground does not depend to any great extent on the number and spacing of struts or anchors, they very much influence the pattern of ground movement to be expected in a given situation. The depth of diaphragm wall/sheet pile determines both the stability of the system and the ground movement associated with it. Depending on the subsoil stratification one may get adequate stability against bottom heave by having the diaphragm wall extended to a stiffer layer existing no more than a few metres below the cut depth. This may not, however, give adequate fixity to the diaphragm wall to minimise ground settlement. The soil structure interaction of braced cuts, therefore, plays a dominant role in determining the overall behaviour and effectiveness of the support system.

The essential features of the design of braced cut in soft ground may be enumerated as follows :

Depth of Diaphragm Wall

There is no established procedure for determining the depth of diaphragm wall below the excavation. In a homogeneous clay not much is gained by taking the diaphragm wall below a critical depth, given by,

$$N_c = \frac{\gamma_H}{C_u} \tag{1}$$

where γ = unit weight of soil;

C_u = undrained shear strength below the cut;

N_c = stability number (≈ 6).

The depth of sheet pile / diaphragm wall is often determined by balancing the moment at the bottom strut level due to active and passive earth pressure on either side of the wall. This gives rise to extended depth of wall, particularly if there is no appreciable improvement of shear strength within the depth of wall. On the other hand it may be adequate to determine the depth of diaphragm wall from consideration of bottom heave alone. If, in particular, the shear strength of the soil improves within reasonable depth below the bottom of the cut it may just be sufficient to take the diaphragm wall to the stiffer stratum. For the entire stretch of Calcutta metro construction, 10-14 m deep excavations have been made with 600 mm diaphragm walls taken to only 4-6 m below the bottom of the cut to rest in a stratum of stiff clay or medium/dense sand, as shown in Fig. 12. Where the diaphragm wall terminates in clay factor of safety against bottom heave may be determined from the expression,

$$F = \frac{C_{u4}N_c + \gamma_3 D_2 + \gamma_4 D_f + \sum C_u \left(H + D_2 + D_f\right)/D_1}{\sum \gamma \left(H + D_2 + D_\gamma\right)} \qquad (2)$$

taking into consideration the contribution of shearing resistance at the soil-wall interface, Fig. 20. A factor of safety of 2.0 would normally suffice. Needless to say, presence of struts – whatever the number – does not contribute to safety against bottom heave.

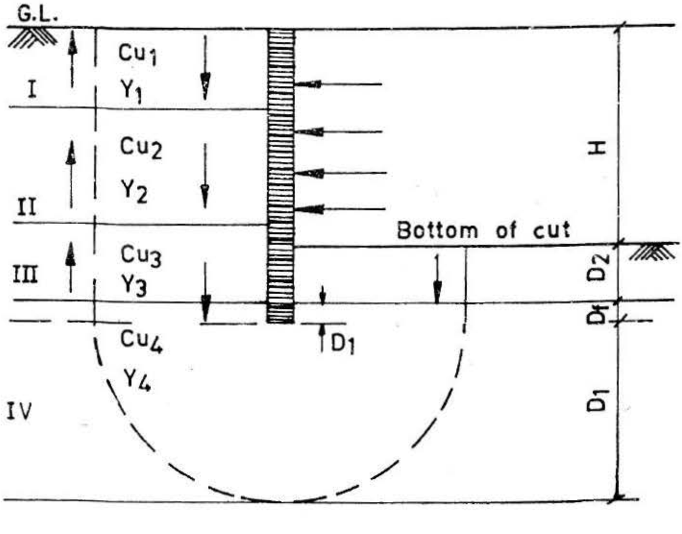


FIGURE 20 : Stability Against Bottom Heave in Stratified Soil

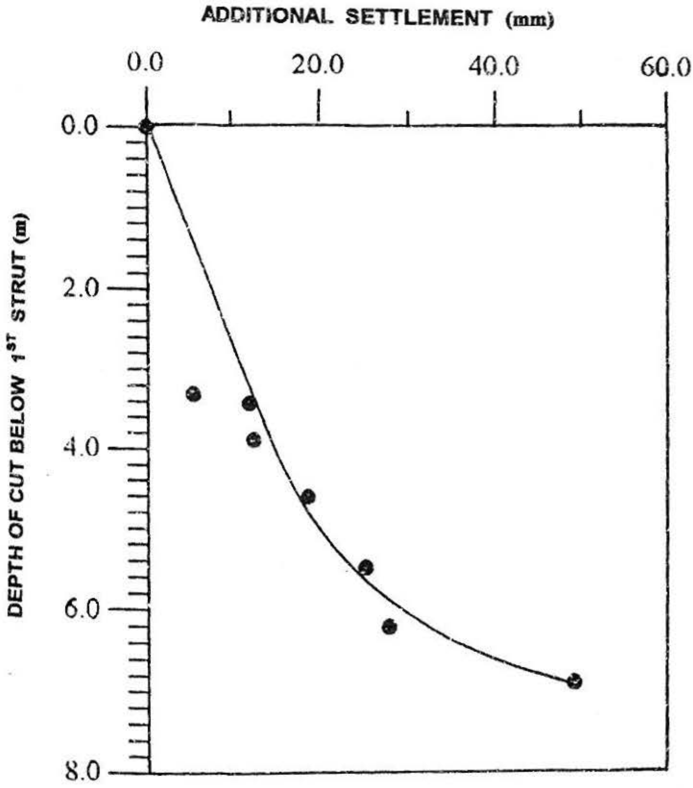


FIGURE 21 : Ground Settlement for Excavation below First Strut

Number and Spacing of Struts

Struts are required to prevent failure of the diaphragm wall in flexure and to minimise lateral deflection of the wall. The diaphragm wall and the struts make up a rigid structural system which prevent excessive ground movement. Obviously, greater the number of struts better is the rigidity of the system. On the other hand, too many struts create obstruction to the construction work underground. In order to study the effect of strut spacing the ground settlements that occurred for the excavation between first and second strut installations were measured. The data are shown in Fig. 21. Settlement values are taken for the total excavation done below the 1st strut level prior to placing the 2nd strut. It can be seen that the settlement increases rapidly for unsupported cut depth of more than 4m. Suitable specifications were incorporated in the design on the basis of these findings. Available case histories also suggest an optimum strut spacing of 3-4 m for deep excavations in soft clay with 500-600 mm diaphragm walls.

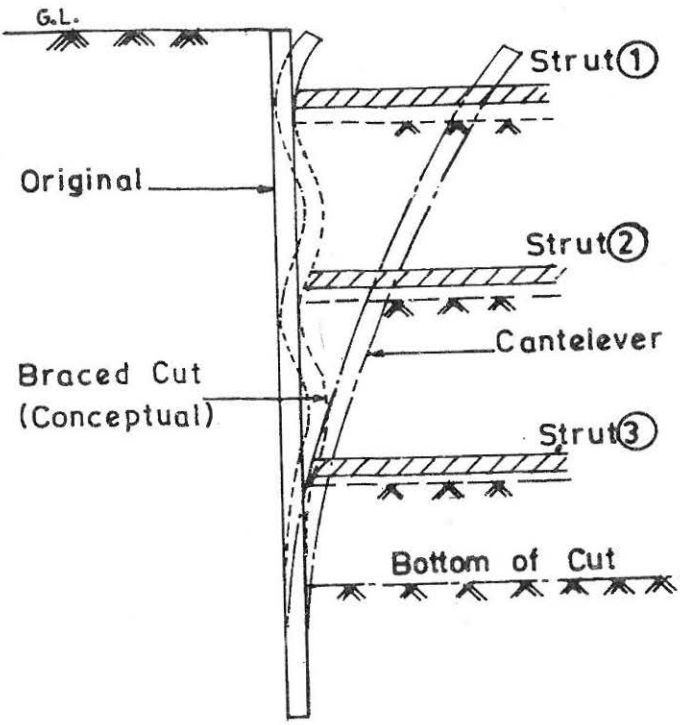


FIGURE 22 : Wall Deflection in Sequential Excavation and Strutting (Conceptual)

Movement of Diaphragm Wall

The movement of diaphragm wall in braced excavation gives rise to a complex soil-structure interaction problem. The very nature of construction sequence involving successive excavation and fixing of struts on the diaphragm wall gives rise to conceptual deflection pattern which is highly indeterminate, Fig. 22. At any stage of excavation wall movement is restricted by the presence of struts. As the excavation approaches the final depth the shallow overburden on the cut (passive) side allows considerable movement of the wall below the cut. Greater depth of penetration inevitably introduces greater degree of fixity in the diaphragm wall below the cut and thereby affect the ground settlement as well as the pressure distribution on the wall.

Earth Pressure

One of the most intriguing aspects of the behaviour of braced cuts in cohesive soil is the distribution of earth pressure on the diaphragm wall.

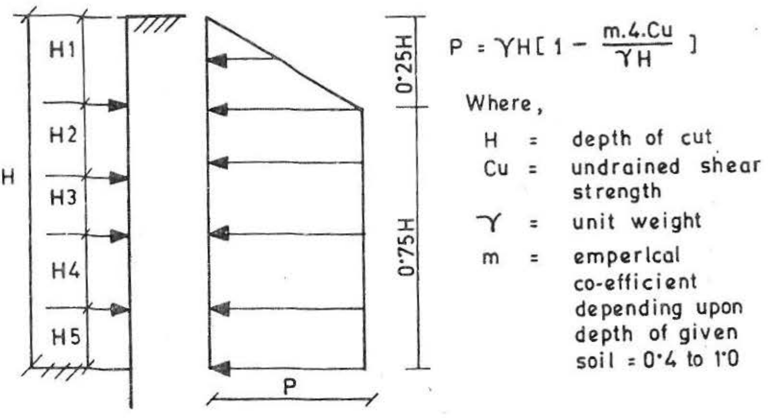


FIGURE 23 : Apparent Earth Pressure Diagram in Braced Cut (Peck, 1969)

This has important effect on the structural design of both the diaphragm wall and the struts. Obviously, the distribution of earth pressure on either side of the diaphragm wall corresponds the K_0- stresses before the excavation commences (Som and Raju 1989). The first stage of excavation allows the diaphragm wall to move freely as a cantilever although the rigidity of the wall may not allow the earth pressure to come down to the active value by the time the first strut is placed - normally, 2-3 m below G.L. Thereafter, the movement of diaphragm wall is restricted by the presence of the strut and stress concentration occurs in the vicinity of the strut as excavation is done below. Similar phenomenon occurs at the second and subsequent strut levels when excavation is continued below the respective struts. The earth pressure that develops is greatly dependent on the strut spacing and the rigidity of the diaphragm wall itself. At all stages, however, it is to be expected that the total earth pressure on the wall would be greater than the active pressure corresponding to that depth. Peck's apparent earth pressure diagram, Fig. 23, is generally used to determine the earth pressure distribution in braced cut (Peck 1969).

Strut Load

Estimation of strut load in braced excavation is a rather complex problem. The strut load depends primarily on the rigidity of the diaphragm wall, spacing of struts and, of course, the soil parameters at a given site. Obviously, the strut load would be a function of the lateral earth pressure that develops on the diaphragm wall at any stage of excavation. But the latter is a highly indeterminate phenomenon - being

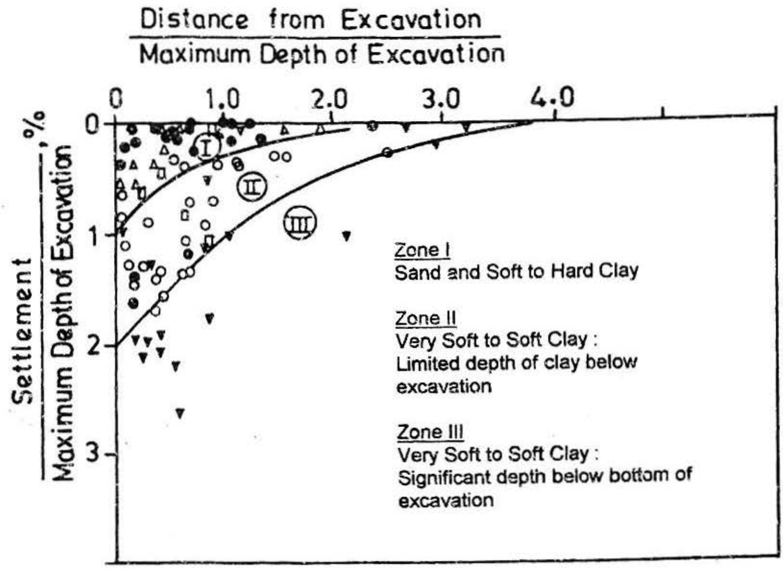


FIGURE 24 : Ground Settlement for Braced Cut with Sheet Pile Support (Pack, 1969)

primarily a function of the interface soil structure interaction. Based on extensive measurement of strut load in the Chicago subway, Peck (op cit) proposed the apparent earth pressure distribution, mentioned above, for the estimation of strut load. However, the actual strut loads that are going to develop in a given situation may only be characteristic of the type of wall, construction methodology, speed of construction, manner of placing the struts etc. Nevertheless, an empirical procedure as proposed by Peck still appear to be the best way of estimating the strut load for initial design.

Ground Settlement

Ground settlement is the surface manifestation of the subsoil deformation in a braced cut. Both the magnitude of settlement and the zone of influence are of importance in determining if adjacent buildings are going to be adversely affected by the construction. The variation of ground settlement for the final cut level may be obtained from Peck's normalised plot for braced cut with sheet pile supports in different types of soil, Fig. 24. The different zones of settlement proposed by Peck are shown as zones I, II and III for different soil conditions below the cut (Peck, op cit).

Instrumentation

Design of braced excavations is still an empirical science and no established analytical procedure is, as yet, available to include the effect of all the parameters to be considered for design. There are, in general, four possible modes of failure of braced excavation, viz.,

(i) excessive movement of the wall,
(ii) yielding of supporting struts,
(iii) bottom heave in cohesive soil and
(iv) piping in granular soils/bursting in cohesive soils, Fig.25 (Winterkorn and Fang 1972).

The permissible movement of wall varies from one site condition to other. In open space appreciable wall movement can often be permitted. But in Metro construction, excavation has necessarily to be made close to existing buildings, pipe lines, etc. which are liable to get damaged even by slight wall movement. Bottom heave may occur due to the bearing capacity failure of the excavation bottom while piping in sand/bursting in clay is caused by excessive hydrostatic

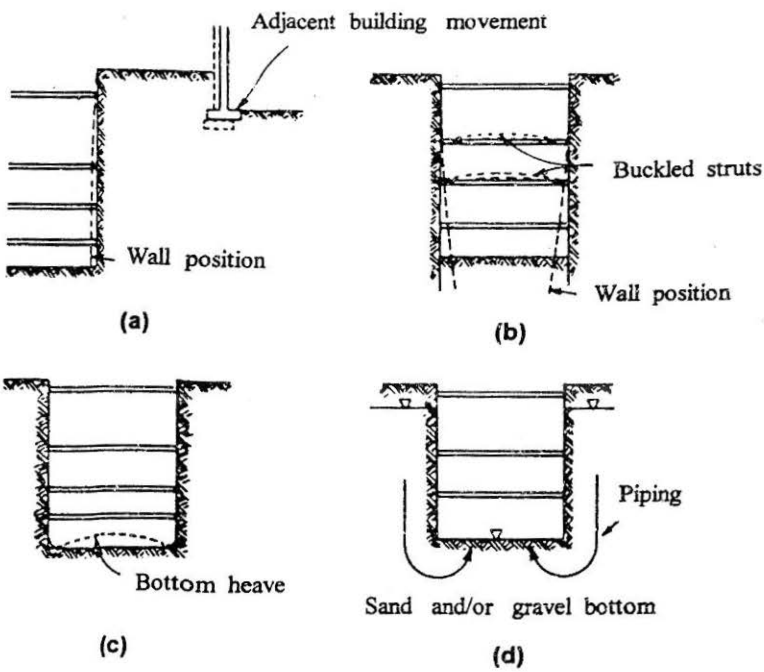


FIGURE 25 : Braced Cut – Modes of Failure

pressure in the water bearing strata below the bottom of the cut. Design of strutted excavation, therefore, involves :

a) Choosing a suitable section for the diaphragm wall/sheet pile and determining its depth of penetration,

b) Determining the position of struts and choosing appropriate sections for the struts.

c) Ascertaining the ground movement caused by the excavation and evaluating its effect on adjoining structures,

d) Working out a programme of dewatering, if necessary.

The objective of an instrumentation programme in braced cut is to investigate the earth pressure developed on the diaphragm wall, ground settlement caused by excavation and placing of struts, diaphragm wall movement and strut load in order to check the structural safety and stability of the cut.

Instrumentation Scheme

Three test sections between Girish Park and M.G. Road stations were selected for extensive instrumentation. The location of instrumented Test Sections in Calcutta Metro alignment is shown in Fig. 26. The first

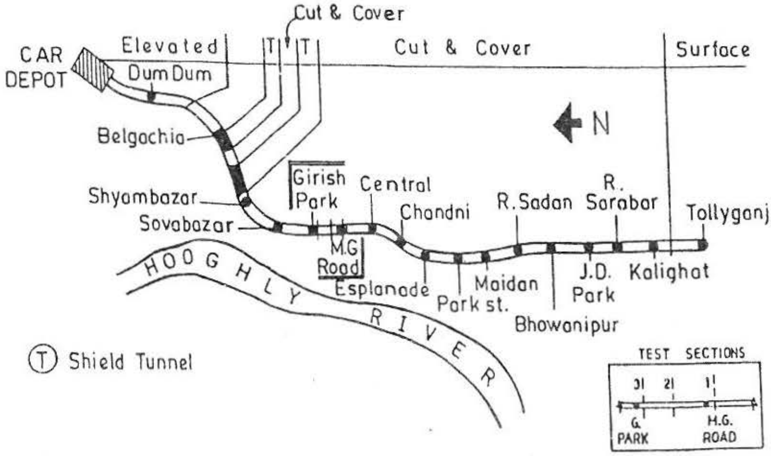


FIGURE 26 : Calcutta Metro Cut – Location of Instrumented Test Sections between Girish park and M.G. Road Stations

Test Section was selected opposite Moti Seal College, the second Test Section near Mahajati Sadan and the third Test Section at Girish Park, all between M.G.Road and Girish Park Stations. The soil strata and excavation profile at Test Section 2 is shown in Fig. 27. A 13.6 m deep excavation was made with four struts spaced @ 3 mm c/c longitudinally. Each Test Section was instrumented for measuring earth pressure on diaphragm wall, pore pressure variation in the soil, ground settlement, wall movement and strut load. Instruments used for these measurements included magnetic settlement gauges for ground settlement, Demac gauges for strut loads, inclinometer (vibrating wire) for diaphragm wall movement, earth pressure cells (vibrating wire) for lateral pressure and both Casagrande and vibrating wire piezometers for pore pressure at different depth, Fig. 28 shows the typical layout of instruments at the test section near Girish Park.

The instruments at each test section included 12 nos. vibrating wire earth pressure cells (GEONOR - AIMIL make) which were fixed to the reinforcement cage of the diaphragm wall - six on the earth face above the cut and three on either face below the cut, Fig. 29. Three vibrating wire piezometers were also placed at 12m, 16m and 20m depths. In addition ten Casagrande piezometers were installed.

Installation Procedure

Inclinometers : Inclinometer pipes (G.I.) were installed within the diaphragm wall during casting of the latter. The G.I. pipes were tied to the reinforcement cage at predetermined locations taking care that the pipes extended well above the top of the reinforcement cage. After the concrete was set around the G.I. pipes in diaphragm wall, the top of the pipe was kept covered with a suitable cap which was taken out from time to time for taking measurements.

After the G.I. pipes were cast in the diaphragm wall, a hollow square aluminium section into which the inclinometer probe would go down subsequently, was inserted into the G.I. pipe and the annular space between the G.I. pipe and aluminium section was grouted with cement. During measurement with the inclinometer, the probe was slid into the aluminium section with the help of pulleys on the outside of the probe. Initial readings at regular intervals of depth were taken in the digital readout unit.

The reading just before the commencement of excavation was taken as initial reading considering the diaphragm wall to be vertical. Subsequently the readings of the probe were taken at different stages of excavation and the difference between the reading at any particular stage and initial reading gave the deflected shape of the diaphragm wall.

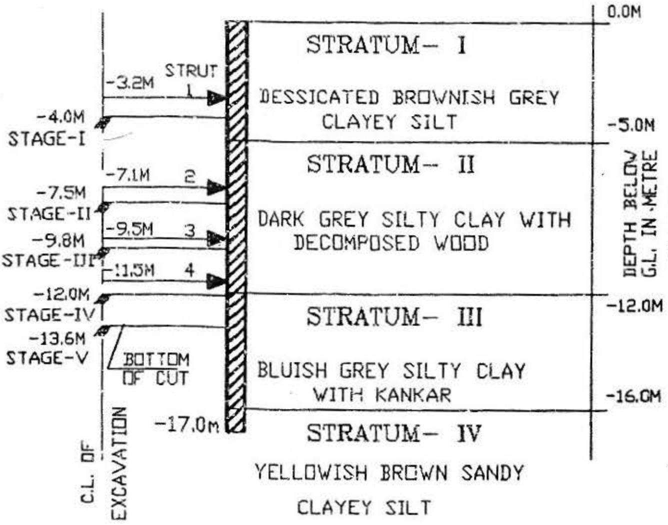


FIGURE 27 : Soil Strata and Excavation Profile for Test Section 2

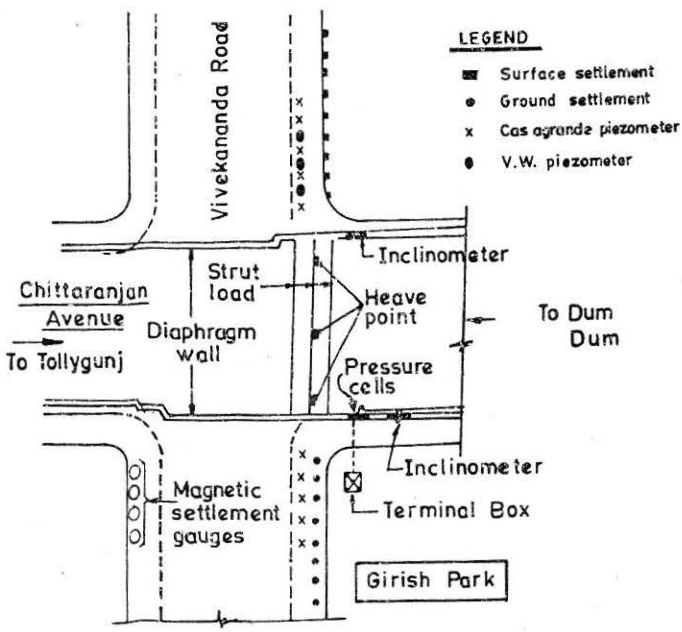


FIGURE 28 : Layout of Instruments in Test Section 3

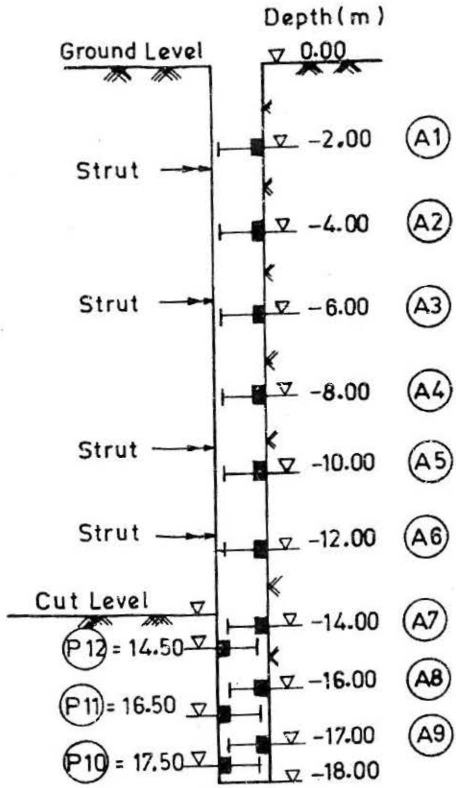


FIGURE 29 : Location of Earth Pressure Cells in Diaphragm Wall

Earth Pressure Cells : Each earth pressure cell / vibrating wire piezometer was calibrated under fluid pressure prior to installation. Fig. 30 shows the calibration curve for a typical earth pressure cell. Special installation procedure was adopted to make the cells come into contact with the soil after lowering the reinforcement cage in the diaphragm wall trench before starting the concreting. Each cell was placed inside a housing, Fig. 31, which was welded to the cage at the appropriate level and operated through a remote - controlled hydraulic jack through a cable and valve manifold on the ground.

While the diaphragm wall trench was being made, the cell housings were welded to the reinforcement cage on the ground and each housing was connected to the valve manifold near the cage top. Continuous monitoring of the pressure cells was done at all stages of installation - starting from fixing the cells to the cage, lowering the cage through the bentonite slurry, operating the jacks to bring the cells in contact with the soil, tremie concreting, setting of concrete, jack release and thereafter. After installation, the lead wires from

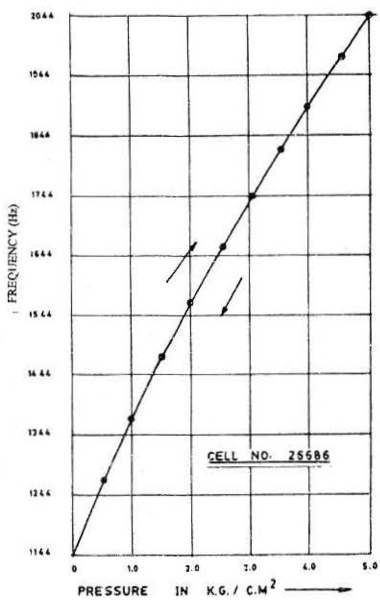


FIGURE 30 : Calibration Curve for a Typial Vibrating Wire Earth Pressure Cell

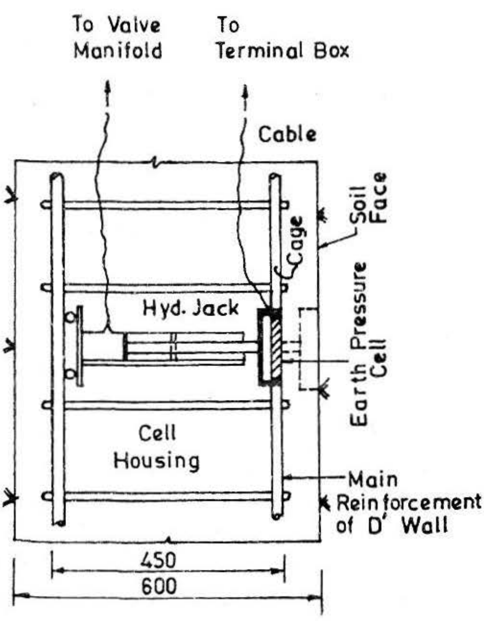


FIGURE 31 : Cell Housing for Installation of Earth Pressure Cell in Diaphragm Wall

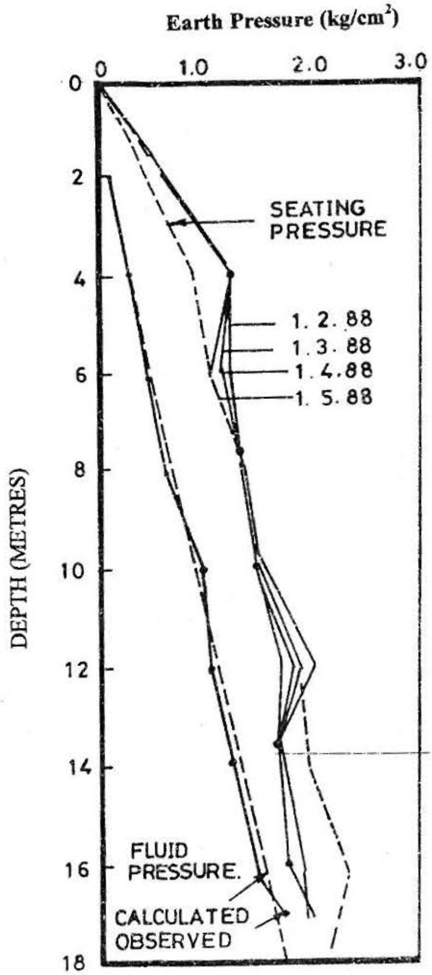


FIGURE 32 : Fluid Pressure Measured by Earth Pressure Cells in Diaphragm Wall Trench before Concreting

the pressure cells were taken in a trench to a terminal box placed inside a nearby building. The vibrating wire piezometers were installed in boreholes of desired depth and the space immediately above the probe was sealed with bentonite plug while the rest of the borehole was backfilled upto the ground surface.

Figure 32 shows the variation of fluid pressure measured by the earth pressure cells immediately after cage lowering at Test Section 1. The calculated fluid pressure for a bentonite specific gravity of 1.05 is shown by

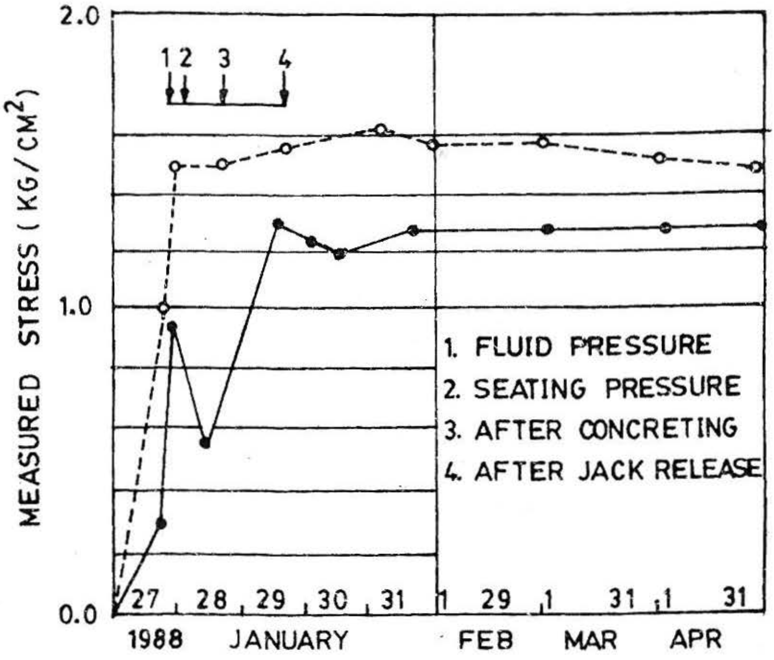


FIGURE 33 : Pressure Cell Readings during Installation

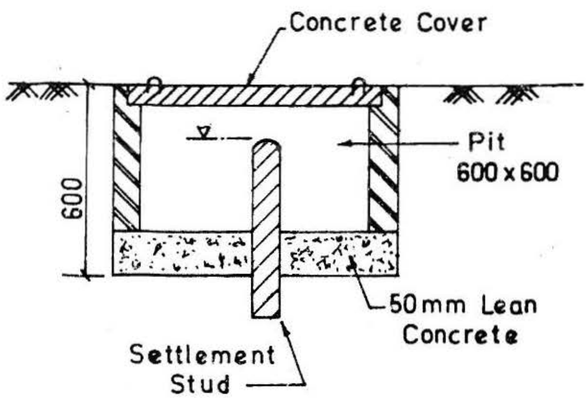


FIGURE 34 : Ground Settlement Measuring Points

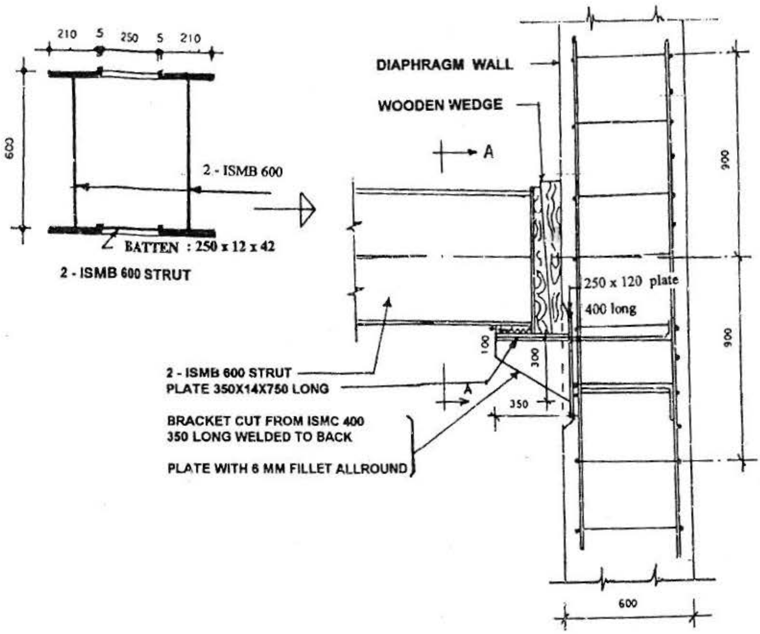


FIGURE 35 : Steel Joists used as Struts

broken lines. The stable pressure cell readings upto four months after concreting of the diaphragm wall are also shown in, Fig. 32. Fig. 33, shows the measured stresses of two typical cells at different stages of installation at Test Section 1. There was some erratic reading during concreting but the pressure tended to stabilize within 7 days.

Ground Settlement : Measurement of ground settlement was made at a number of locations to ascertain the pattern of settlement adjacent to the diaphragm wall. Hydraulic settlement gauges, were used and a reference datum was established at least 60 m away from the wall. Each settlement point consisted of a steel rod embedded about 1 m below the ground in a pit which was kept covered by a concrete slab, Fig. 34. At the time of taking readings the concrete slab was removed and one end of the settlement gauge was held on the steel rod. The levels of the settlement points were taken on different dates and the difference in level between any two dates gave the relative movement of the point between these dates.

Strut Load : The struts were fabricated from two steel joists (ISMB-450/ISMB-500/ISMB-600) with batten plates along the length and two end plates, Fig. 35. The length of a fabricated strut was kept same as the distance between two diaphragm walls at the particular location. During the

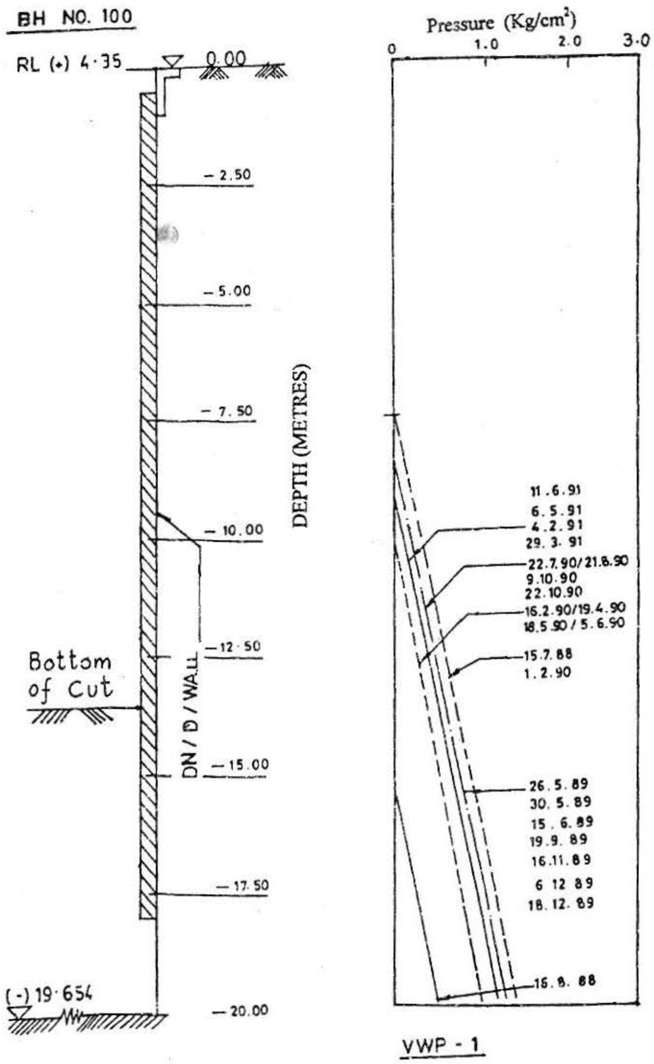


FIGURE 36 : Pore Pressure Distribution aas Measured by Vibrating Wire Piezometers

installation, the struts were placed on brackets fixed on the diaphragm wall and preloaded through wedge to 10-20 t by hydraulic jacks. For determining strut load, stainless steel Demac pins were fixed on the struts at pre-determined locations at a gauge distance of 200 mm. Subsequently, the distances between pins were measure by extensometers at regular intervals of time. At any stage of excavation, therefore, the strain in the joists could be

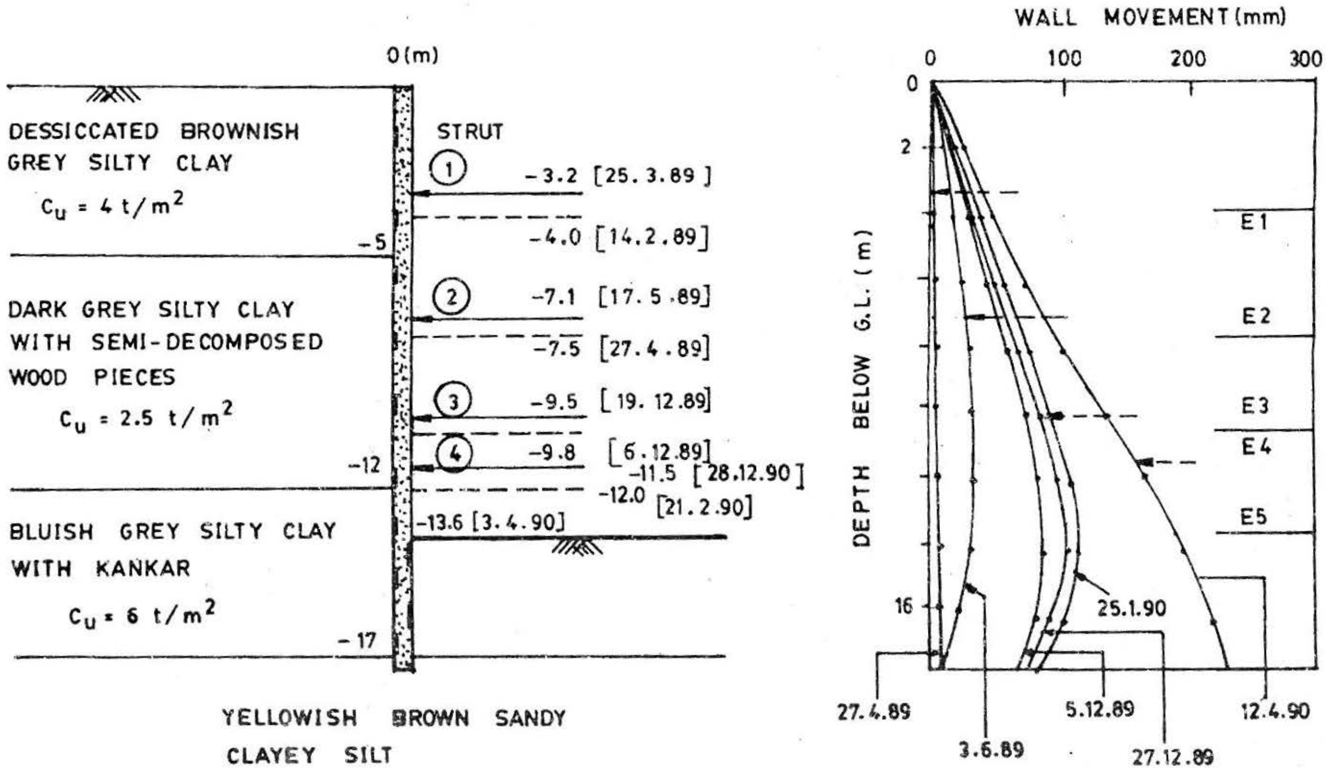


FIGURE 37 : Diaphragm Wall Deflection at Different Stages of Excavation

obtained and the same converted into load. The effect of temperature variation was separately studied and the measured strut loads were suitably corrected.

Pore Pressure : Pore pressure was measured with the help of vibrating wire and Casagrande piezometers installed at certain distance away from the diaphragm wall. Fig. 36 shows the pore pressure distribution on different dates at Test Section 2. There was no major fluctuation of pore water pressure during the period of construction.

Performance Study

Movement of Diaphragm Wall

Figure 37 shows the inclinometer data at Test Section 2. The wall deflection increased with the depth of cut but, the rate of increase was more as the depth increased. At any stage of excavation the wall movement above the cut was restricted by the presence of struts while free movement was allowed below the cut. As the excavation approached the final depth the shallow overburden on the cut (passive) side allowed considerable movement of the wall below the cut. With only 4 m penetration of the diaphragm wall below the bottom of the cut the maximum wall deflection increased from 100 mm to 225 mm for the final stage of excavation from 12 m to 13.6 m. A greater depth of penetration of the diaphragm wall would possibly have restricted the wall movement somewhat by introducing a greater degree of fixility to the wall. Lim et al. (1991) report a wall deflection of only 120 mm for a 17 m cut in soft marine clay with 1200 mm diaphragm wall going 26 m below the bottom of the cut.

Earth Pressure

Before Excavation : The lateral pressure distribution as measured by the pressure cells on different dates after installation are shown in Fig. 38. The initial seating pressure in each cell could be more than the in-situ earth pressure - or less - but the imbalance tended to even out with time and the stable earth pressure indicated by the pressure cells gave the in-situ lateral stresses in the soil.

It may be seen that the distribution of lateral pressure does not follow a definite pattern with depth. The pressure cells at 2 m depth in Test Section 2 might not have made adequate contact with soil prior to concreting because of apparent overcutting near the top. Three cells did not show any pressure other than the fluid pressure, initially - but later, they started to show increasing pressure. The observed stresses could still be less than the in-situ stresses because of yielding of the soil. Furthermore, when compared with the

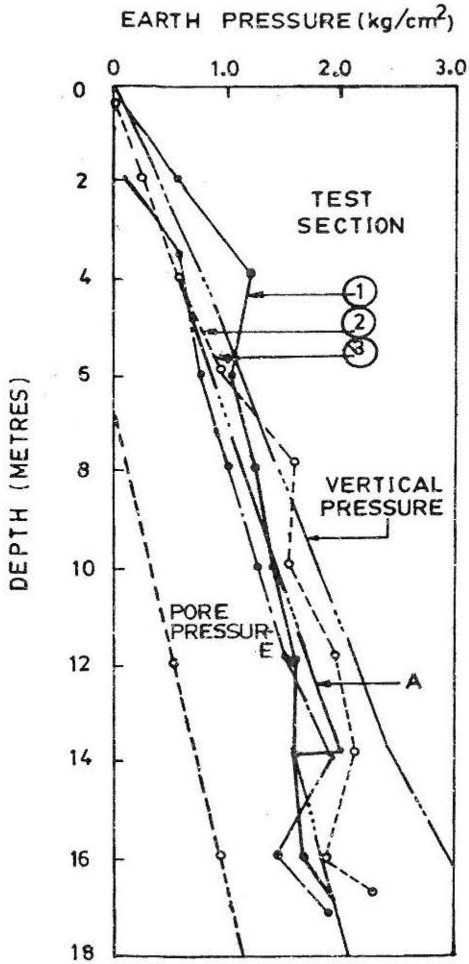


FIGURE 38 : Earth Pressure on Diaphragm Wall before Commencement of Excavation

distribution of overburden pressure the lateral earth pressure in Test Section 1 is, in fact, higher than the vertical stress near the ground surface. Fig. 38 also shows the pore water pressure at different depths as measured by the vibrating wire piezometers in Test Section 1. The piezometric head is generally found to be 7 m below the ground surface.

The distribution of vertical and lateral stresses and the pore water pressure with depth, as shown in Fig. 38, make it possible to obtain by back calculation the K_0 - value of the soil at different depths. The data are shown

Table 2. Values of in - situ K_0

Depth (m)	Location 1	Location 2	Location 3	Average
2	1.65	0.77*	0.55*	1.0
4	1.70	0.85	0.80	
6	0.95	0.85	0.73	
8	0.83	1.16	0.70	0.70
10	0.70	0.84	0.65	
12	0.67	0.90	0.60	
13.5	0.50	0.80	0.70	
16	0.37	0.48	0.30	0.50
17.5	0.44	0.65	0.45	

* Low values due to inadequate seating.

in Table 2. Notwithstanding the low values of K_0 obtained at 2 m depth at locations 2 and 3 for reasons given earlier the average K_0 appears to correspond with the change of soil characteristics for the different strata. On the basis of the observed data the average K_0 value for the three significant strata may be taken as 1.0, 0.7 and 0.5 respectively.

During Excavation : Figs. 39.1 and 39.2 show the distribution of measured stresses on the diaphragm wall at different stages of excavation. It clearly shows major concentration of stresses in the vicinity of the struts. Fig. 40, summarises the data for all the significant stages of excavation. An attempt has been made in Figs. 39.1 and 39.2 to compare the measured stresses above and below the cut with the corresponding active and passive earth pressures and Peck's apparent earth pressure distribution for each stage of excavation. The following points emerge from this comparison :

a) Above the cut depth the earth pressure gradually increases with depth and is always greater than the corresponding active pressure.

b) At any stage of excavation the earth pressure increases with depth but remains essentially constant beyond a certain depth.

c) The earth pressure below the cut tends to approach the active pressures on earth side while the same approaches the passive pressure on the cut

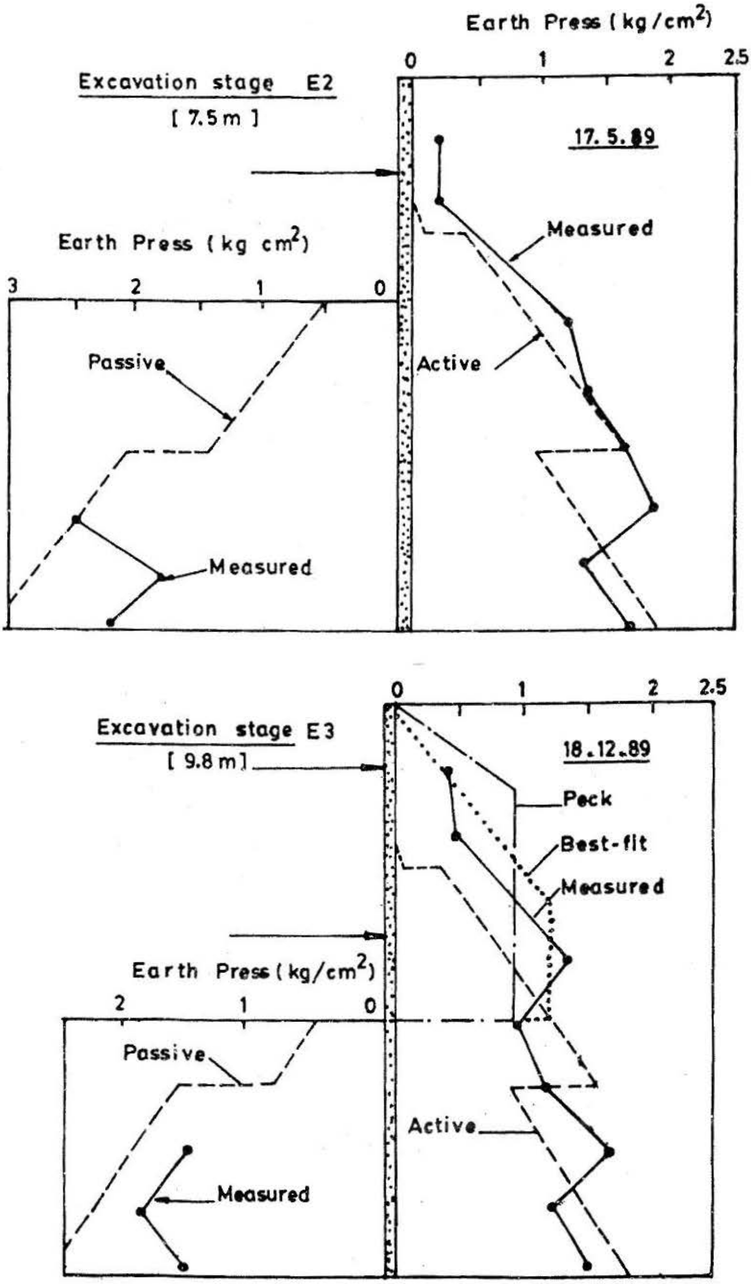


FIGURE 39.1 : Measured Stresses on Diaphragm Wall at Different Stages of Excavation

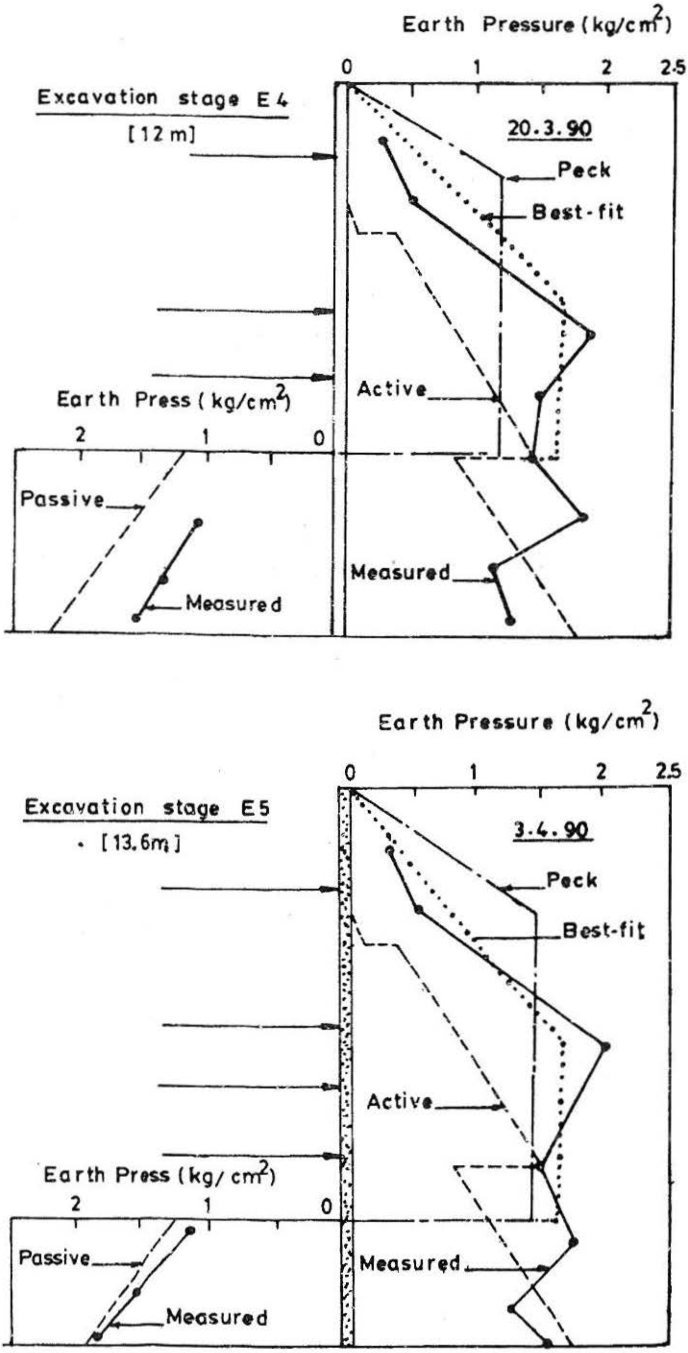


FIGURE 39.2 : Comparison of Measured Stresses with Peck's Apperent Earth Pressure Diagram

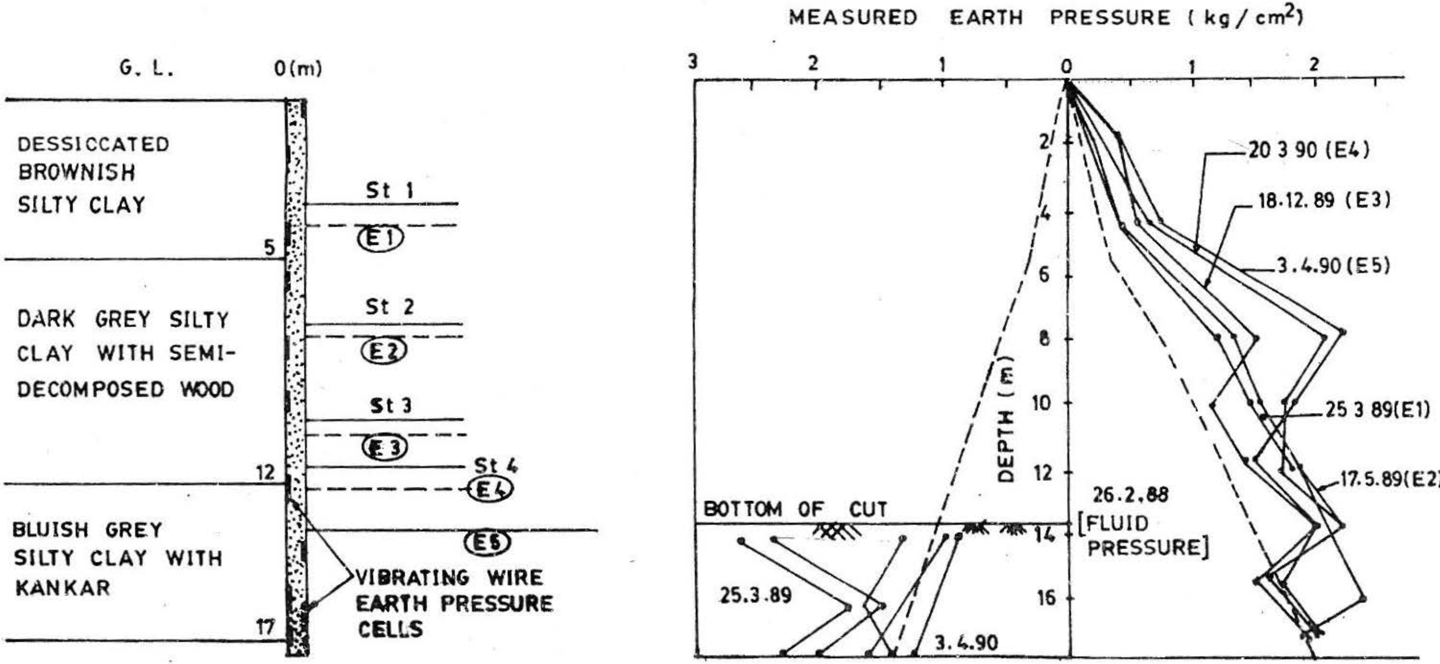


FIGURE 40 : Summary of Measured Earth Pressure at Different Stages of Excavation

Table 3 : Comparison of Earth Pressure Obtained by Peck's Method and the Observed Data

Depth of cut (m)	Method pressure	Maximum earth (t/m^2)	m	Depth at which linear variation changes to constant pressure (m)
10.0	Peck	10.8	0.80	2.5
	Observed	12.5	0.55	6.0
12.0	Peck	12.0	1.00	3.0
	Observed	17.0	0.50	7.2
13.6	Peck	15.0	1.00	4.4
	Observed	18.0	0.65	8.2

side. In fact, at the final cut level, the earth pressures on either side of the wall come very close to the active and passive pressures. This appears consistent with the wall movement which had reached as high as 5.6% of the depth of wall below the cut. For greater depth of diaphragm wall, however, the situation could be different because smaller relative wall movement would not allow the active and passive pressures to be mobilised below the cut.

It is interesting to compare the measured stresses with the apparent earth pressure obtained by Peck's empirical analysis. A best fit trapezoidal earth pressure diagram has been drawn with the observed data for each stage of excavation. For the purpose of calculation, the value of coefficient 'm' has been taken as 0.8 to 1.0, as obtained from an earlier analysis of strut loads in Calcutta metro construction (Som and Ghosh 1985). While the general trend of Peck's trapezoidal distribution appears to be valid, the depth at which the linear variation of pressure changes to constant pressure occurs at approximately 0.6H instead of 0.25 H. The value of coefficient 'm' also appears to be different. Not much need, however, be read in this discrepancy because the data represent the observation for the Calcutta soil only. Further, the first two struts were placed at a greater distance apart (3.9 m) as compared to the lower struts (2.0-2.4 m) in Test Section 1. This would have allowed more deformation of the diaphragm wall in the upper part. Table 3 gives a comparison of the maximum apparent earth pressure as obtained by Peck's method and the pressure obtained by fitting a traphezoidal distribution to the observed data.

Strut Load

Table 4 gives a comparison of the measured strut load with loads

Table 4 : Calculated vs. Measured Strut Load

Depth of cut (m)	Measured Strut Load				Peck's Method					Best Fit Diagram				
	1	2	3	Total	1	2	3	4	Total	1	2	3	4	Total
10.0	36.3	37.3	–	73.6	42.6	36.7	–	–	79.3	28.2	52.2	–	–	80.4
12.0	43.3	43.3	21.7	108.3	44.4	38.4	28.8	–	111.6	32.0	49.6	40.8	–	122.4
13.6	–	–	–	–	52.5	48.0	31.5	22.5	154.5	29.6	47.7	37.8	27.0	142.1

calculated by Peck's method and the best-fit trapezoidal diagram. Both Peck's method and the observed pressure diagram overestimate the strut load by 5-15 % but the results are predicted well for practical purposes.

The variation of measured strut load with depth at two typical sections - in North and South are shown in Figs. 41 and 42. Although it was to be expected that after placing of second or third strut the load in the first strut would not increase appreciably with further excavation this was not always the case, although the expected pattern was noticed in contract section 15C. Still, it can be generally said that once a strut was placed, further excavation causes major increase in load in that strut - the earlier struts being less affected by further excavation. In addition to the instrumented test sections the strut loads on 170 struts in 10 contract sections were measured. In some locations all the struts on the same vertical plane were observed together to get an indication of the total load on the exposed portion of the diaphragm wall. Fig. 43 gives a compilation of the observed data The total strut load has been plotted against the depth of excavation to represent the load on the diaphragm wall above cut level for 3 m length of wall. The figure reveals that the load increases slowly for shallow depth of cut, i.e. upto 7 m. Thereafter the load builds up much faster with depth. Notwithstanding the wide scatter of data an average curve may be drawn to represent the variation of total load on the exposed height of diaphragm wall with depth

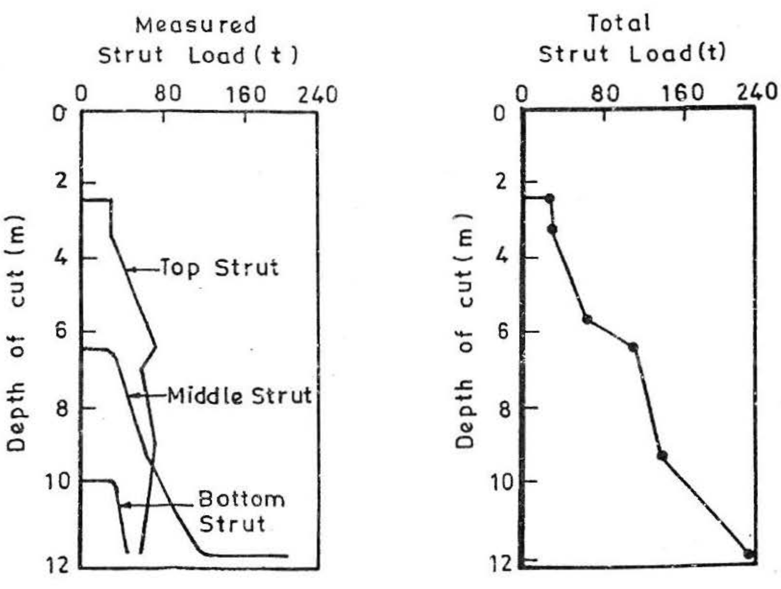


FIGURE 41 : Variation of Strut Load with Depth of Excavation – Section 15

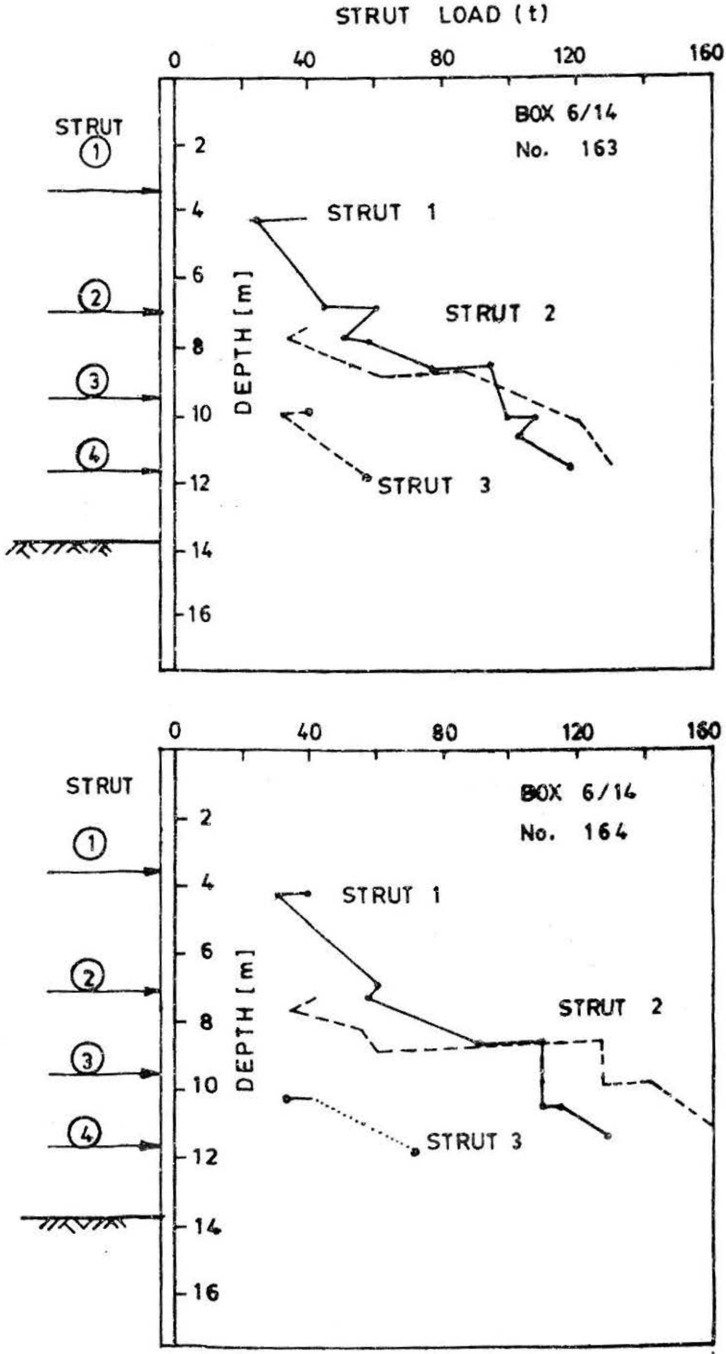


FIGURE 42 : Variation of Strut Load with Depth of Excavation – Section 16

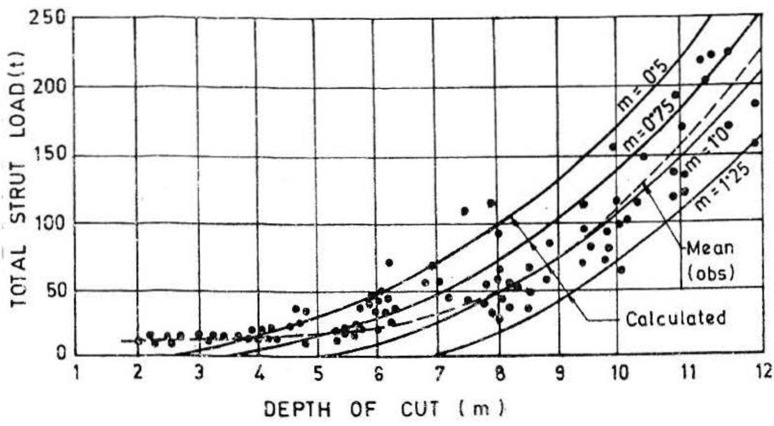


FIGURE 43 : Total Strut Load Vs. Depth of Excavation

of cut. For example, the total load increases from 25 tonnes per metre length of the wall for a 7 m cut to 50 tonnes per metre for a 12.5 m depth of cut.

In order to examine the equilibrium of diaphragm wall under the lateral forces a study has been made of the strut loads, the apparent earth pressure on the diaphragm wall and active / passive earth pressure below the cut. The results are shown in Table 5.

The data indicate fairly good correlation of the forces on either side of the diaphragm wall at equilibrium. It may, therefore, be concluded that the apparent earth pressure diagram above the cut and the active and passive earth pressures below the cut represent fairly well the distribution of earth pressure on the diaphragm wall. This may, however, be only true for small depth of wall below the cut for which the Calcutta metro data are strictly valid. For greater depth of wall below the cut, effect of fixity may alter the pressure distribution below the cut although it is unlikely that the apparent earth pressure distribution above the cut will be significantly affected.

Ground Settlement

Ground settlement is the surface manifestation of the subsoil deformation in a braced cut. Both the magnitude of settlement and the zone of influence are of importance in establishing if adjoining buildings are going to be adversely affected by the construction. Fig. 44 shows the settlement profile at different stages of excavation in Test Section 1. In order to get a clear picture of the variation of ground settlement, the settlement data for the final cut level for different test sections are plotted in non-dimensional terms,

Table 5 : Forces on Diaphragm Wall

Depth of Cut (m)	Total Strut Load (measured) (t) P_1	Load of Passive Earth Pressure (t) P_2	Total Load on Cut Side (t) $P_3 = P_1 + P_2$	Load of Apparent Earth Pressure Above Cut Depth (t) P_4	Load of Active Earth Pressure (t) P_5	Total Load on Earth side (t) $P_6 = P_4 + P_5$	Ratio $\frac{P_3}{P_6}$
10.0	80.4	137.5	219.9	99.0	112.6	211.6	1.04
12.0	122.4	95.5	217.9	142.8	84.0	226.8	0.96
13.6	142.3	61.0	203.3	171.0	63.0	234.0	0.87

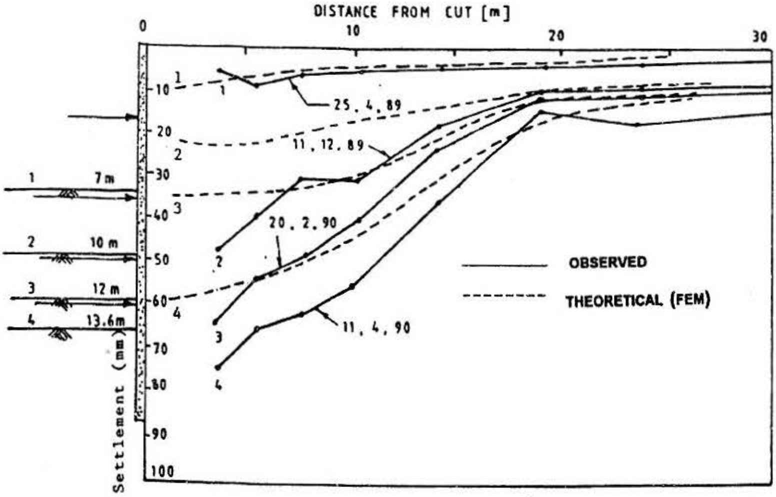


FIGURE 44 : Ground Settlement Profile at Section 1

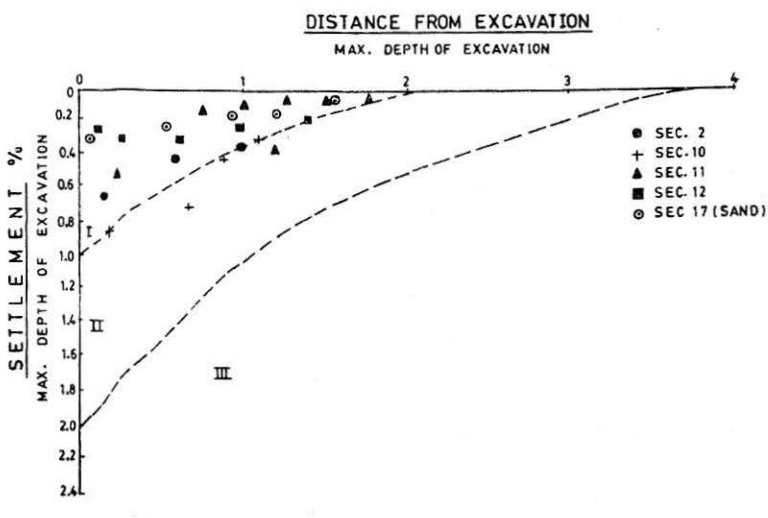


FIGURE 45 : Variation of Ground Settlement at Different Test Sections

Fig. 45. A curve passing through the upper limits of the settlement values gives the maximum settlement adjacent to the cut and its variation with distance from the cut. It may be seen that the maximum settlement adjacent to the cut has not exceeded 1% of the depth of cut, and, even for the upper limit of the settlement profile, the zone of influence does not appear to extend beyond a distance of three times the depth of cut.

The settlement profile as obtained for the measured sections may be compared with Peck's normalised plot for braced cut with sheet pile supports in different types of soil. It may be observed that the measured settlements fall well within zone I of Peck's diagram whereas, on the basis of soil condition in the Calcutta metro, it should have fallen in zone II. This leads to the conclusion that measured settlements have been considerably less than those predicted by Peck's diagram. This appears to be the result of using the somewhat stiff diaphragm walls which have greater rigidity than sheet piles. Fig. 46 gives the variation maximum ground settlement with depth of cut.

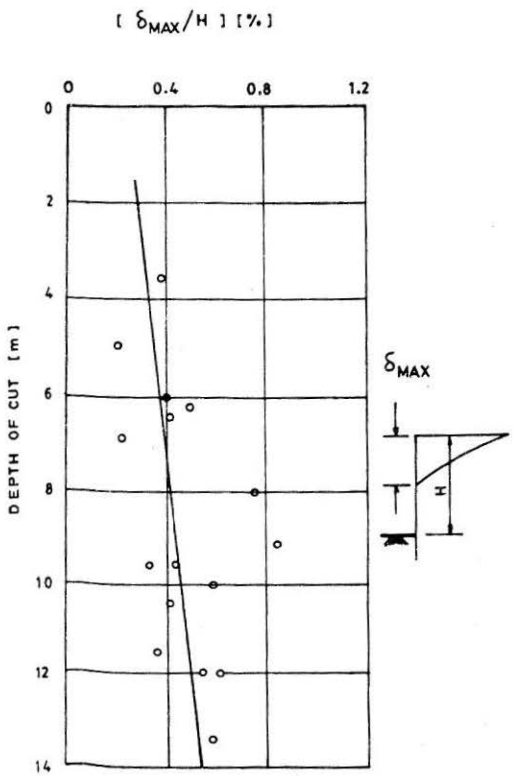


FIGURE 46 : Maximum Ground Settlement Vs. Depth of Excavation

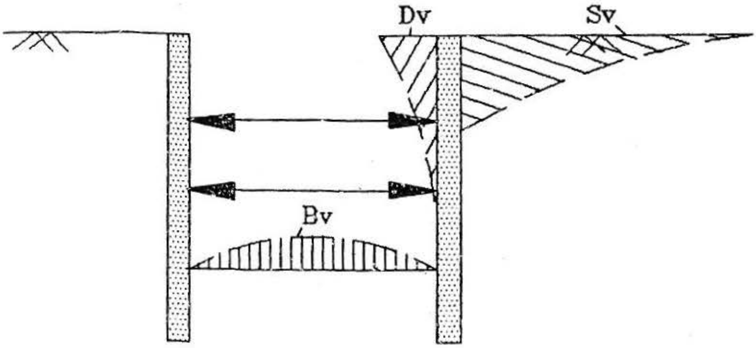


FIGURE 47 : Volume of Lost Ground and Wall Deflection

A normalised plot of maximum ground settlement / depth of cut has been made against the depth of cut at different test sections. It may be seen that the data lie, more or less, on a straight line with δ_{max}/H varying between 0.3% to 0.6%.

It is interesting to compare the ground settlement and the corresponding wall movement in terms of the volume of lost ground (S_v) and the volume of soil displaced by the wall above the cut (D_v), Fig. 47. There should, ideally, be a relationship between D_v and S_V if the deformation is truly elastic. Table 6 shows a comparison of S_v and D_v for the Test Sections of Calcutta metro for different depth of excavation.

It is observed that the volume of lost ground can be reasonably predicted from $S_v - D_v$ relationship as suggested by Lim et. al. (1991) from the observations of the Bugis cut.

Table 6 : Ground Loss (S_v) vs. Wall Deflection Volume (D_v)

Depth of Cut	S_v (m^3 / metre)	D_v (m^3 / metre)
10.0	0.6	0.52
12.0	0.8	0.75
13.6	1.2	1.30

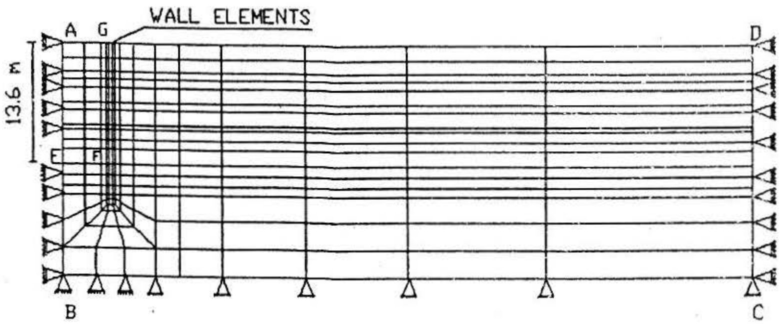


FIGURE 48 : Finite Element Analysis of Braced Cut – Calcutta Metro Tect Section 2

Theoretical Study

An attempt was made to develop a mathematical model for performance evaluation of the Calcutta Metro excavation and to make a parametric study of the effect of different factors on the relevant soil-structure interaction phenomena. Plane strain finite element analysis of the instrumented test sections was done, Fig. 48. Sequential excavation and strut installation in the stratified soil was considered to simulate the field conditions. Eight-nodded isoparametric elements, using three-point Gauss quadrature rule for reduced integration, were used to model the soil and the wall. Each node consisted of two degrees of freedom in the form of in-plane displacements one in the horizontal direction and the other in the vertical direction. Instead of using 'slip' or joint elements in between the soil and the wall, thin layer of soil elements were used in the interface. The installation of the diaphragm wall, prior to excavation, was assumed to have little effect on the in situ stresses in the soil. Effective stress analysis was done considering undrained behaviour of the soil. Two different models of soil nonlinearity, namely, Modified Cam-clay model and Drucker-Prager elasto-perfectly plastic model were used.

The process of excavation was simulated by removing the elements row-wise in front of the wall and thereafter making the excavated surface stress free. The struts were considered as springs. The spring stiffness of each strut was determined from the sectional properties of the strut and the horizontal spacing of the struts. Each load increment was spread over a number of equal sub-increments to ensure numerical stability and to achieve convergence. An equilibrium check was done at each stage of loading and the out-of balance loads were adjusted with the subsequent load increments.

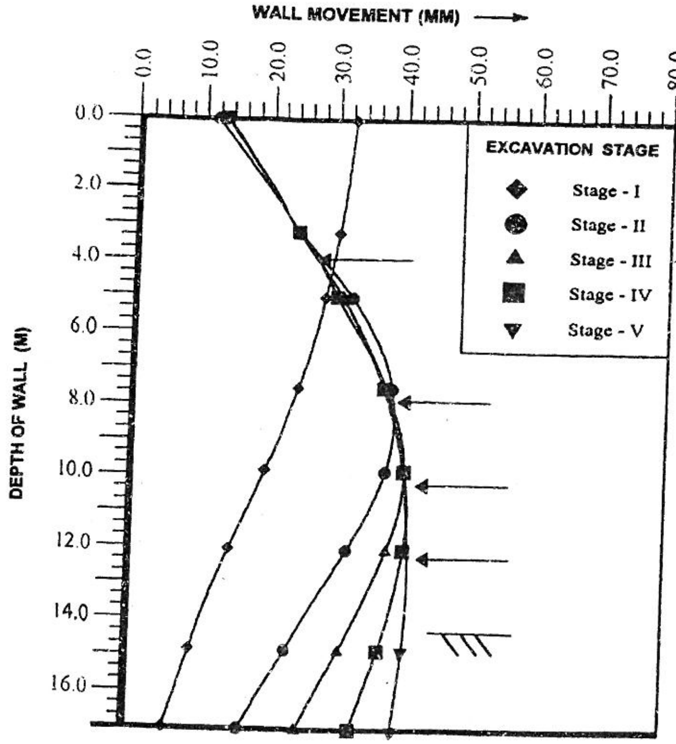
The subsoil profile and excavation scheme used for the theoretical analysis of Test Section 2 of the Calcutta Metro cut are shown in Figure 27. Different stages of excavation and strut installation have been considered in the analysis. For example, stage-II in the above figure indicates excavation of soil down to 7.5 m below the G.L. after installation of strut 1. The full excavation was done in five stages. The final depth of cut was 13.6 m below Ground Level.

The diaphragm wall deflection and ground settlement as obtained for each stage of excavation are shown in Figs. 49.1 and 49.2 for the linear analysis and Figs. 50.1 and 50.2 for the nonlinear analysis considering the modified Cam-clay constitutive relationship for the soil. The results show the same patterns of wall and ground movements in both linear and nonlinear analysis but larger ground of movement when soil is treated as materially nonlinear.

The load in each strut is evaluated from the spring force, by multiplying the strut stiffness by the relative displacement of the wall at the point of strut application. Fig. 51 shows the load in each strut per metre of excavation. It clearly indicates that the force in each strut increases till the installation of the subsequent strut and the lower struts are most heavily loaded. The total earth pressures as obtained from FE analysis and as observed in the field for the final stage of excavation are presented in Fig. 52. The apparent earth diagram of Peck and the best fit plot of the field data are also superimposed in Fig. 52. It can be seen that the theoretically predicted earth pressure lies well within Peck's envelope but is far from being trapezoidal.

The equilibrium of the diaphragm wall under the lateral forces has also been studied from the strut load and earth pressure distribution on the diaphragm wall, Fig. 53. Table 7 gives the magnitude of all the forces per metre length of wall at the final cut level, as obtained from the theoretical predictions and as calculated from field data. In both cases, the results indicate fairly good correlation of the forces on either side of the diaphragm wall although the total forces obtained from theoretical analysis and field measurements differ by 20-25%.

The vectors of accumulated displacement for the final cut level is plotted in Fig. 54. The figure confirms that soil movement takes place essentially in the form of ground settlement, wall deflection and bottom heave. The soil movement is significantly high towards the bottom of the excavation and towards the wall toe. Maximum heaving of the soil takes place at the centre of the excavation while the maximum settlement in the adjacent ground takes place at some distance away from the wall.



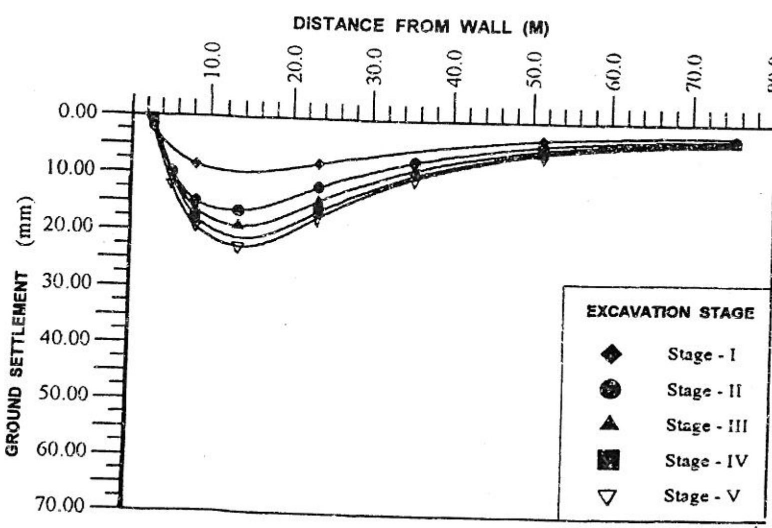


FIGURE 49 : Diaphragm Wall Deflection and Ground Settlement – Linear Analysis

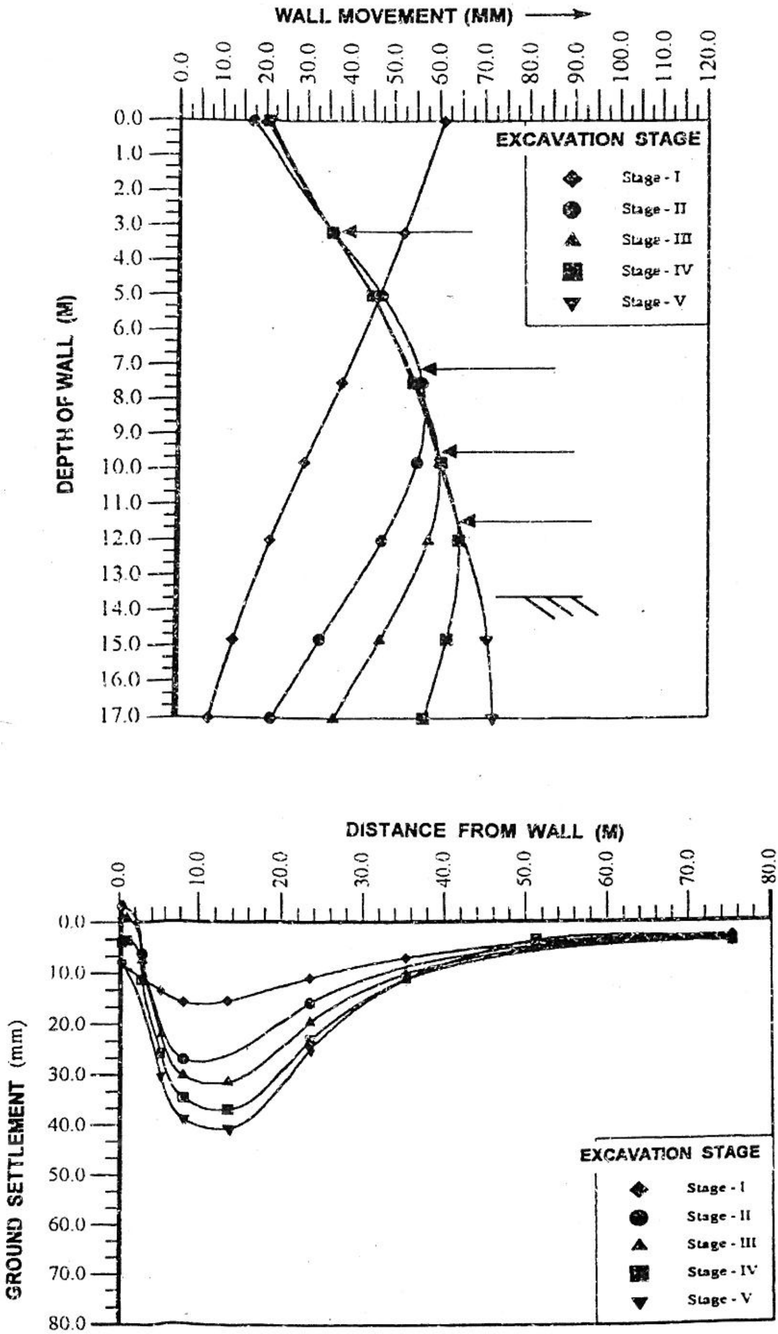


FIGURE 50 : Diaphragm Wall Deflection and Ground Settlement – Non-Linear Analysis

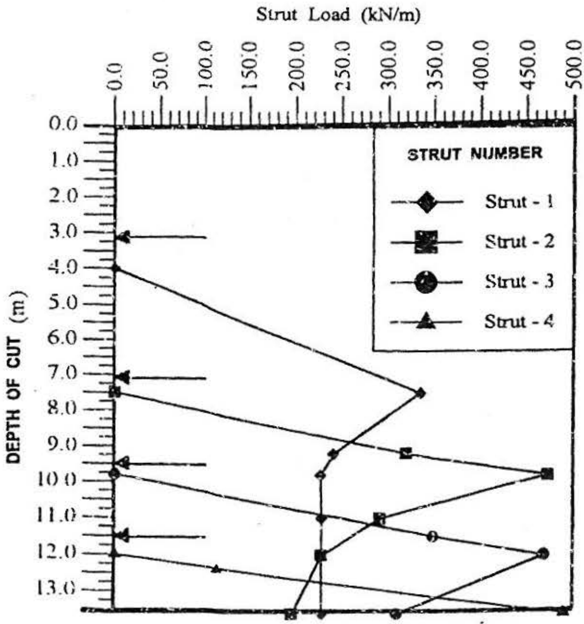


FIGURE 51 : Variation of Strut Load with Depth – F.E. Analysis

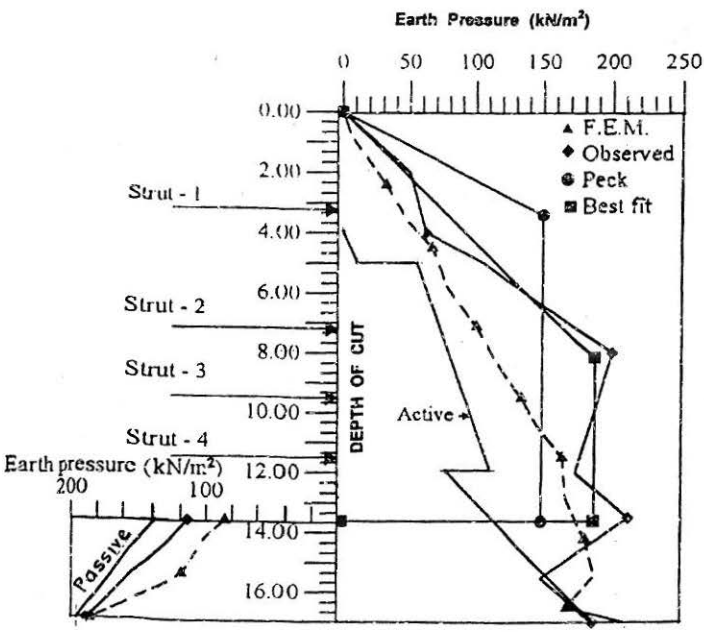
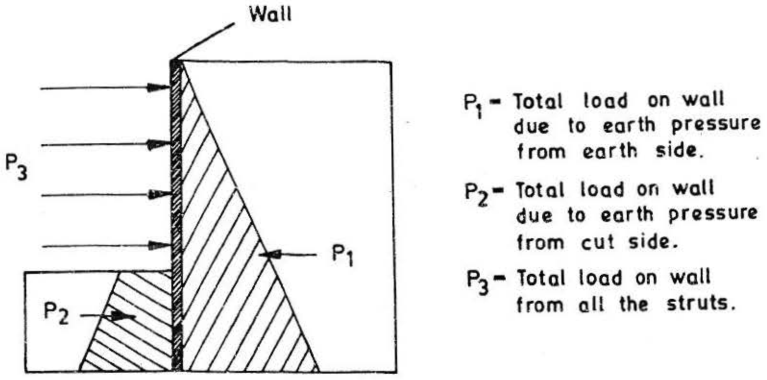


FIGURE 52 : Earth Pressure Distribution from F.E. Analysis



$$P_1 = P_2 + P_3$$

FIGURE 53 : Lateral Forces on Diaphragm Wall – Equilibrium Analysis

Comparison with Field Data

The theoretical values of the wall deflection are compared with the field measurements for different excavation stages, Fig. 55. The general trend of the wall movement is similar to the observed data but the measured values are higher particularly for greater depths of cut where discrepancy is large.

Table 7 Lateral Forces on the Diaphragm Wall at Final Cut Level

Source of Data	Field Observations	FE Analysis (Non-linear using Modified Cam-clay)
Total Load on Earth side (P_1) (kN/m)	2340.0	1897.3
Load due to Earth Pressure on Cut side (P_2) (kN/m)	610.0	492.4
Total Strut Load (P_3) (kN/m)	1545.0	1238.4
Total Load of Cut side ($P_2 + P_3$)	2155.0	1730.8
Ratio $\frac{P_1}{(P_1 + P_3)}$	1.08	1.09

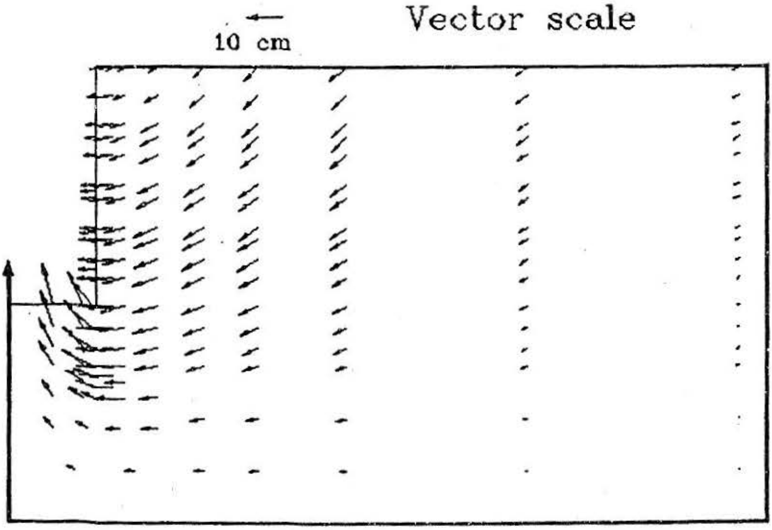


FIGURE 54 : Vectors of Accumulated Displacement at Final Cut Level

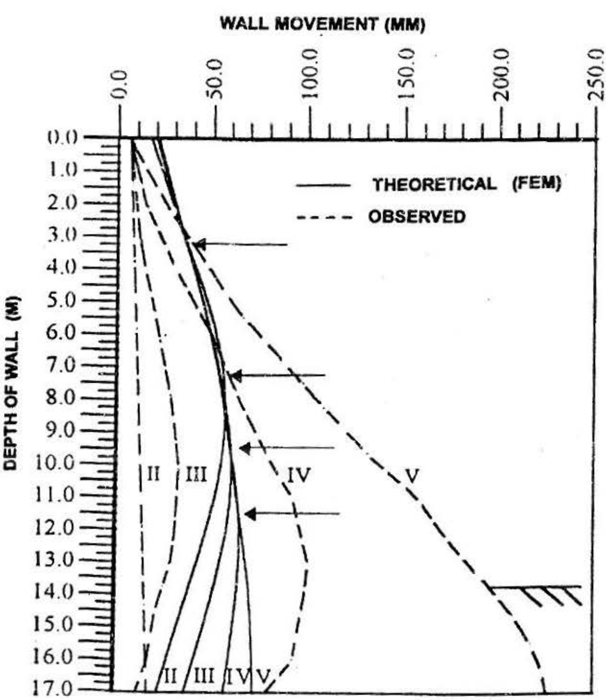


FIGURE 55 : Comparison of Theoretical and Observed Wall Deflection

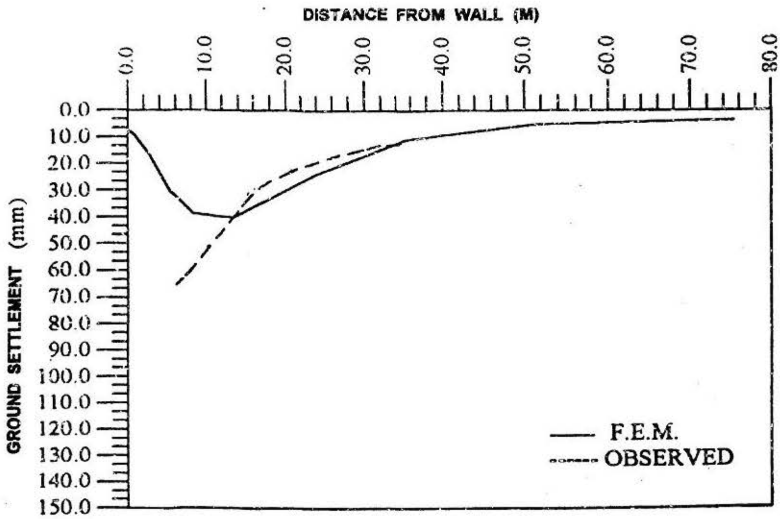


FIGURE 56 : Ground Settlement – Comparison of Theoretical and Observed Data

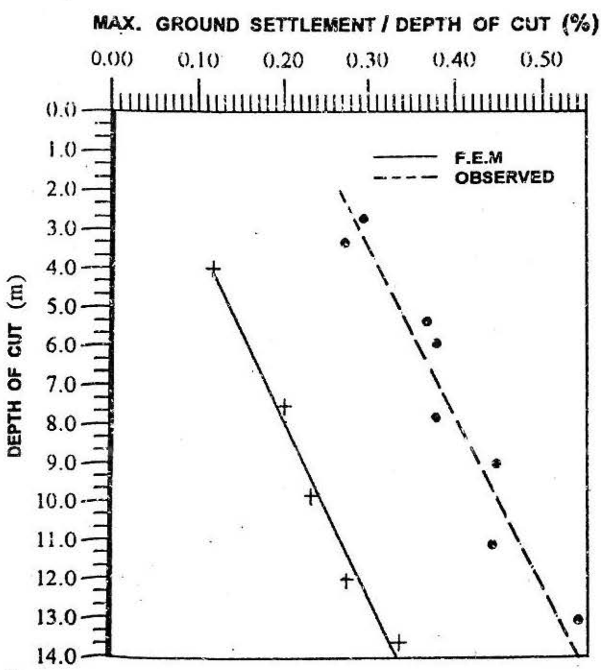


FIGURE 57 : Maximum Geound Settlement Vs. Septh of Cut – Theoretical and Observed Data

Figure 56 represents a plot of vertical ground settlement adjacent to the wall for the final stage of excavation while the best fit plots of maximum ground settlement against depth of cut, as obtained numerically and the field are shown in Fig. 57. In general, the ground settlement is underestimated in the FE analysis. This may be due to the fact that the theoretical analysis was carried out under short term conditions and assumed ideal conditions of construction which seldom prevail in the field. Lack of construction control, improper fixation of struts, delay in strut installation, slow rate of excavation etc. come to play. Moreover, long term effects of creep leads to large wall movements. All these factors would lead to discrepancy between the theoretical and observed data.

Parametric Study

A parametric study of different factors that influence the behaviour of braced cuts in soft clay has been made for the Calcutta metro cut. Some of these studies are reported here.

Penetration of Wall : The performance of a braced excavation depends largely on the depth of penetration of wall below the final excavation level. The more the wall extends below the bottom of cut the more is the fixity of the wall at its base and the deflection of the wall toe gets reduced. For the Calcutta Metro Test Section -2 the stiffer layer starts at a depth of 16.0m below G.L. while the rigid base was found at 26.0 m below G.L.. Accordingly, the diaphragm wall was taken to only 3.4 m below the bottom of cut to rest in the stiff layer. Analysis has been done for the same excavation scheme and same subsoil but considering varying depth of embedment of wall below the cut. The results of analysis for the final stage of excavation are presented in Fig. 58. As the penetration of the wall in the stiffer layer increases, greater fixity develops at the wall toe and the wall deflection at the bottom gets reduced, while the same above the final strut level remains unaltered. The trend of toe movement with increase in wall penetration below the bottom of the cut will be evident from the nondimensional plot, Fig. 59. The deflection of the toe decreases linearly with the increase of wall penetration, and approaches zero at perfect fixity when the movement of the wall is fully restrained. However the increase of wall penetration hardly affects the settlement of ground adjacent to the cut.

Width of Excavation : The influence of excavation width on the soil-wall interaction has also been studied. Excavation for the reference analysis was 10.0 m wide. Three more analysis were carried out for excavation widths of 8.0 m, 12.0 m and 14.0 m. Figs. 60.1 and 60.2 summarise the effect of excavation width on the wall movement and ground settlement for the final cut level. Both wall deflection and ground settlement increase rapidly with the increased cut width. In fact, about 35 mm additional wall deflection

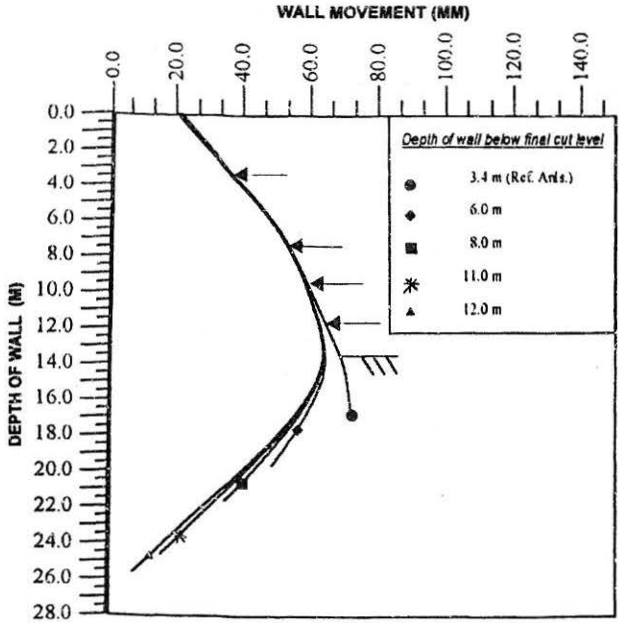


FIGURE 58 : Wall Movement below Cut – Effect of Depth of Embedment

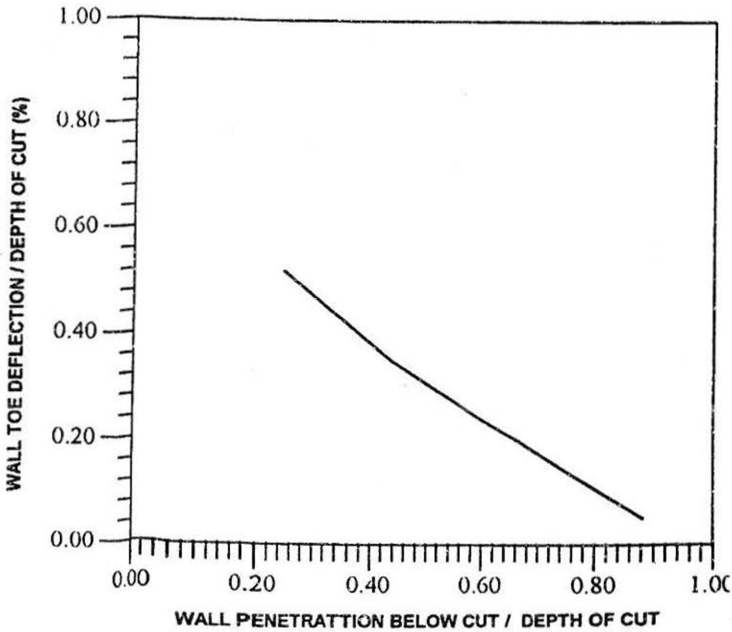
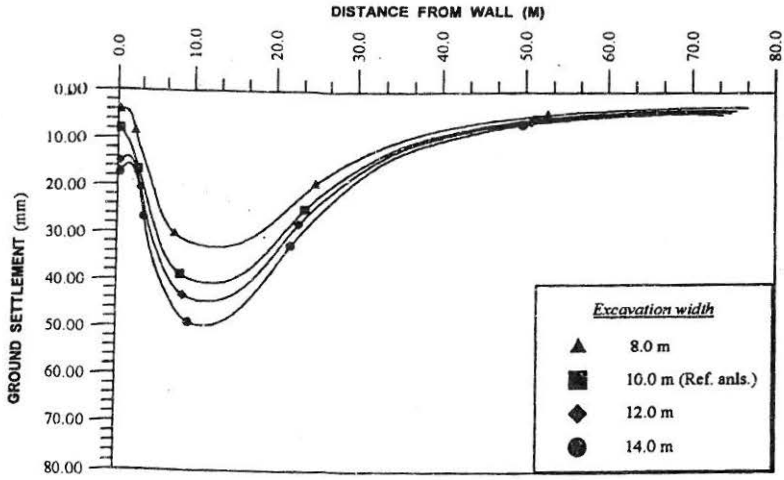


FIGURE 59 : Movement of Toe Vs. Wall Penetration
– Non-Dimensional Plot



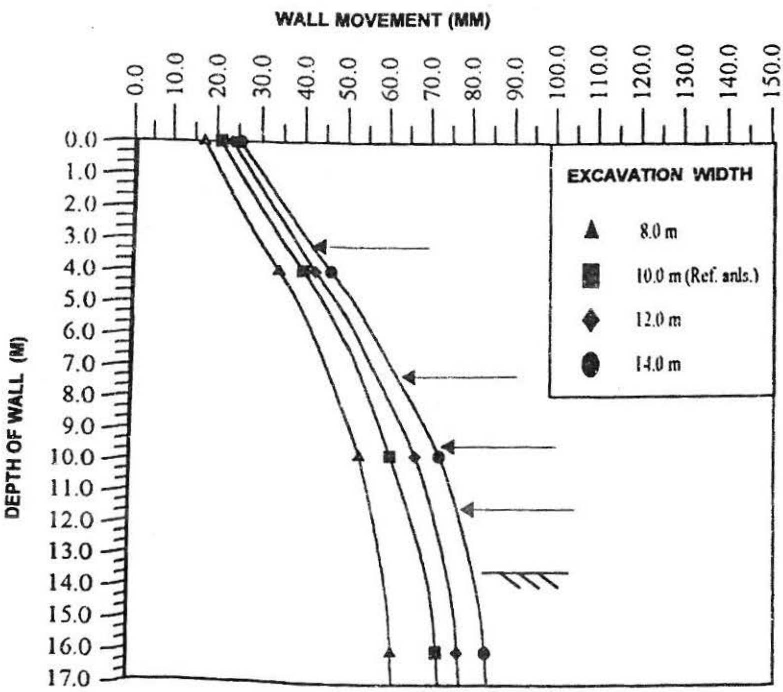


FIGURE 60 : Wall Movement and Ground Settlement
– Efect of Excavation Width

would occur for 14.0 m wide cut as compared to that for the 8.0 m wide cut. A normalised plot of the maximum ground settlement, for the final cut level, at different widths of excavation, Fig. 61, reveals that the former increases almost linearly with the increase in the latter. This is attributed to a larger zone of plastic deformation of soil taking place for a wider excavation. So, to limit ground and wall movement and to maintain stability in case of wider excavations, it may be necessary to provide additional lateral supports.

Strut Spacing : Struts are required to prevent the failure of the diaphragm wall in bending, to minimise the lateral deflection of the wall, as well as the ground settlement. The diaphragm wall and the struts make a rigid structural system which prevent excessive settlement of adjacent ground. Obviously, greater the number of struts, better is the rigidity of the system. On the other hand, too many struts cause problems of obstruction to the underground construction.

A study was made to determine the effect of strut spacing on wall deflection, ground settlement and the forces on diaphragm wall. A number of

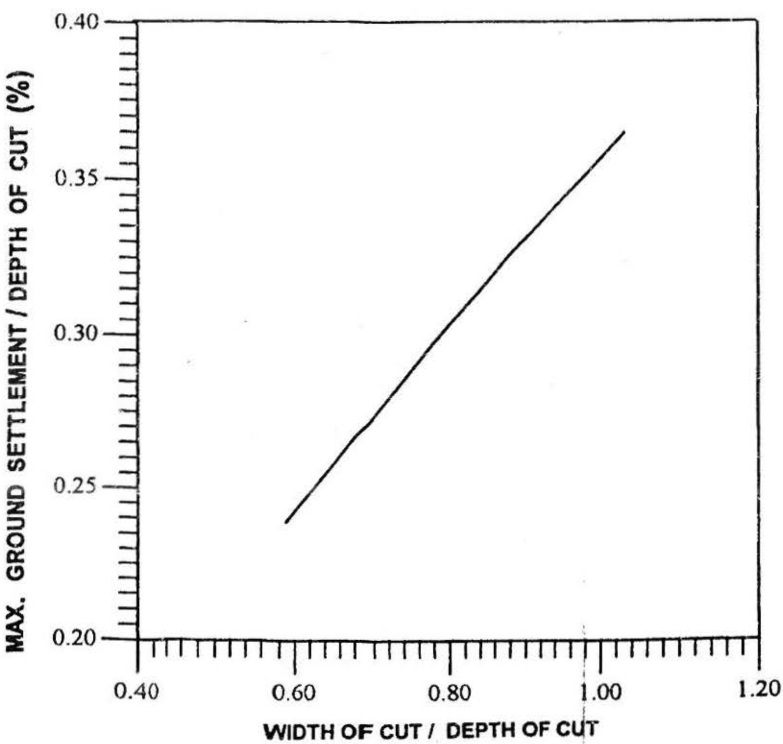


FIGURE 61 : Maximum Ground Settlement Vs. Width of Excavation

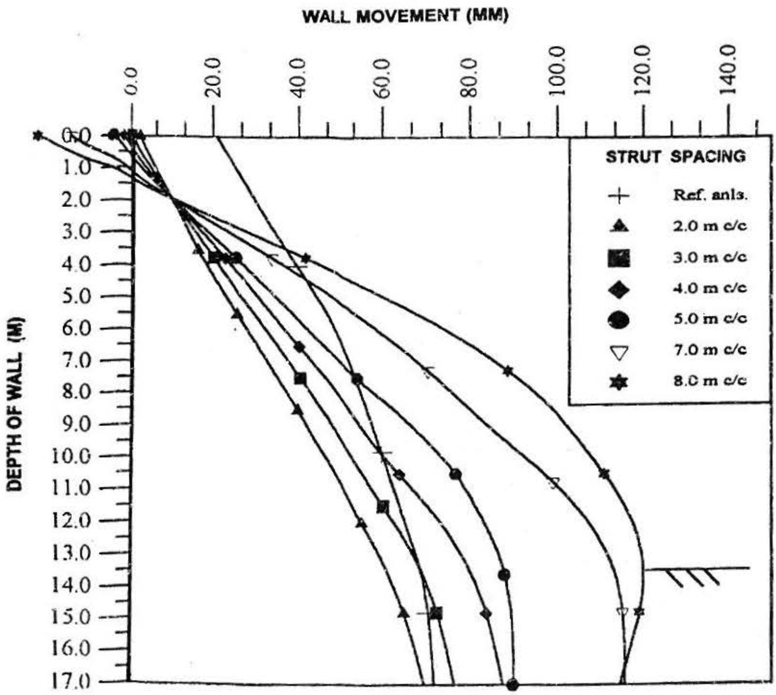


FIGURE 62 : Effect of Strut Spacing on Wall Deflection

FE runs with different vertical spacing of struts were made on the same soil-wall system and for the same final depth of cut as described in the reference analysis. Thus, for 2.0 m spacing of struts, six struts were placed at 2.0 m, 4.0 m, 6.0 m, 8.0 m, 10.0 m and 12.0 m after excavation had reached 2.5 m, 4.5 m, 6.5 m, 8.5 m, 10.5 m and 12.5 m depths respectively. Thereafter, excavation was done to the final cut level of 13.6 m as done for the reference analysis. Similarly, for 3.0 m strut spacing four struts, for 4.0 m and 5.0 m strut spacing three struts and for 7.0 m and 8.0 m spacing only two struts were used.

The wall deflection for different strut spacing is plotted against depth of wall in Fig. 62. Beyond a strut spacing of 4.0 m the movement appears excessive. In fact, for 7.0 m strut spacing an additional maximum displacement of about 40 mm takes place as compared to that of 4.0 m strut spacing. A similar trend of results is also observed from Fig. 63 which shows the ground settlement data for different strut spacing. Here also excessive ground settlement occurs beyond a strut spacing of 4.0 m. To further study the matter the maximum ground settlement is plotted against strut spacing in Fig. 64. The plot appears linear upto a strut spacing of 4.0 m. Beyond this point the curve becomes steep as ground settlement increasesrapidly with strut spacing.

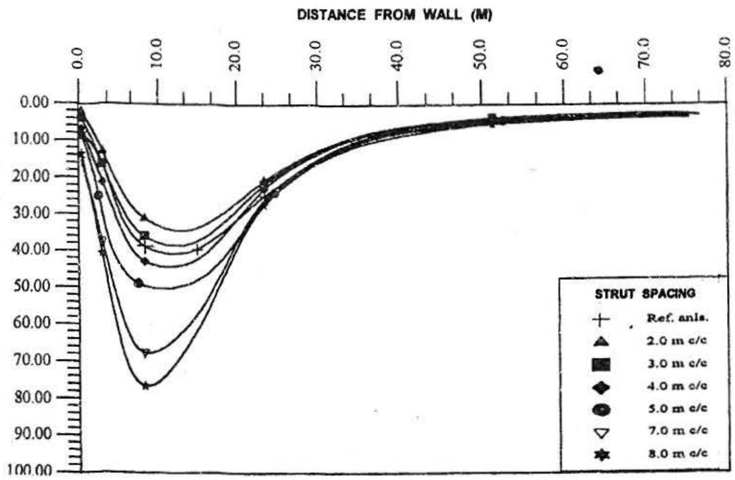


FIGURE 63 : Effect of Strut Spacing on Ground Settlement

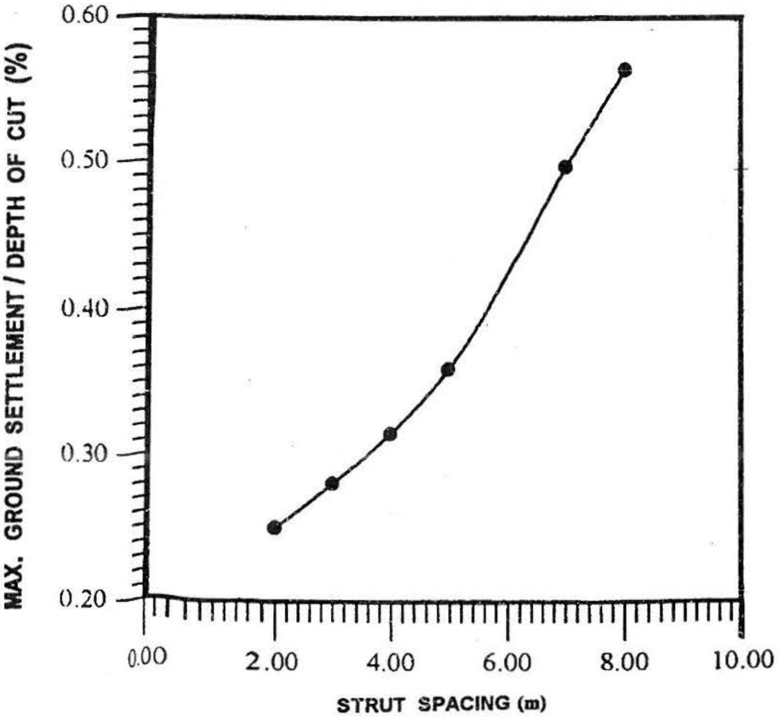


FIGURE 64 : Maximum Ground Settlement Vs. Strut Spacing

Wall Thickness : A parametric study of effect of increasing wall thickness was carried out to investigate its effect on the performance of braced excavation. Increase in wall thickness inevitably means increase in the relative rigidity of the wall. The movement of the diaphragm wall at the final excavation level for different wall thickness is shown in Fig. 65. It is observed that wall thickness does not significantly affect the magnitude of wall deflection. In fact, increasing the wall thickness from 600 mm to 1000 mm reduces the maximum deflection of the wall by only 15 mm. Also, increase in wall thickness does not appear to have any major effect on the ground settlement. It can, therefore, be said that increase in wall thickness does not improve the performance of the braced excavation to a major extent. Similar parametric study of other factors, like strut pre-stressing, soil stiffness and soil anisotropy shows varying degrees of influence on the ground settlement and wall deflection. Of all the factors, however, strut spacing and penetration of wall appear to have the most significant influence on the performance of braced cuts.

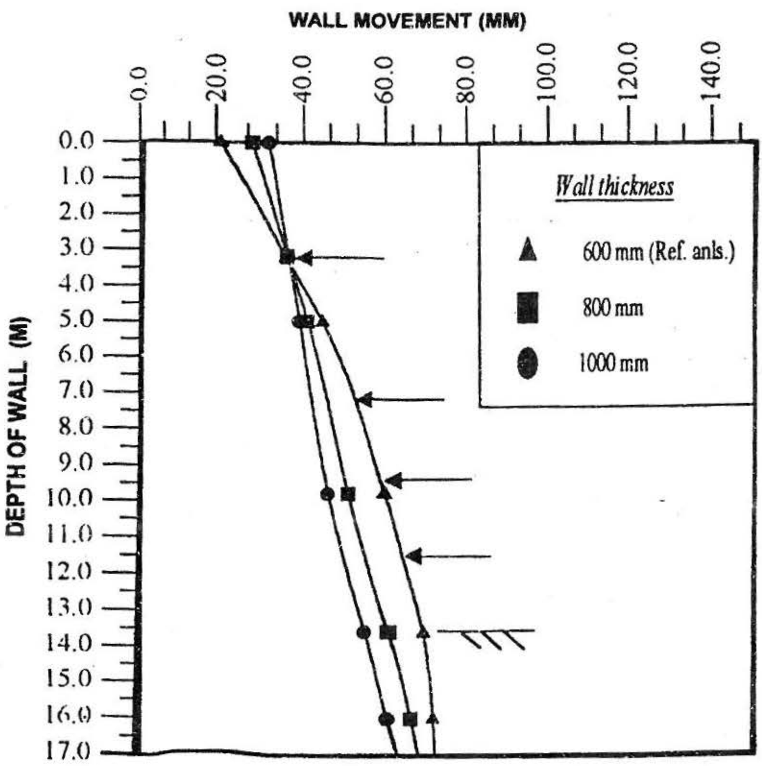


FIGURE 65 : Effect of Diaphragm Wall Thickness on Wall Deflection

Construction Control

The main purpose of construction control in deep excavation is to ensure that

a) Ground settlement does not become so high as to cause damage to adjoining structures.

b) There is no collapse of the side or bottom of excavation due to shear failure of the soil or yielding of support.

Design of braced cut is made to prevent failure of any of the above kinds. However, in the absence of any straight - forward soil structure interaction analysis, design is based on semi-empirical approaches rather that theoretical analysis. Whatever the stable design arrived at, it is invariably based on certain conditions and assumptions regarding field conditions which are supposed to be adhered to at site. If there is any major deviation, which may or may not be immediately apparent, the behaviour of excavation may be drastically different from what has been predicted in design. Therein lies the need for construction control. It essentially means following a construction procedure which satisfies the design consideration as closely as possible.

Effect on Adjacent Buildings

At the outset, a survey of all the buildings on both sides of the Metro alignment had been made. Most of these buildings were of brick masonry construction supported on shallow spread footings. The buildings were generally old with poor maintenance. It had been decided that the buildings would be kept under general observation right form the start of construction. If a building showed any sign of distress at any stage of construction, monitoring of the building would be done on a regular and systematic basis. The types of observations generally undertaken for monitoring the buildings were :-

a) Settlement observation on the plinth/floor of the building. These were done to obtain the settlement profile of the building on different dates. For this, settlement points were marked and settlements were observed with respect to a fixed datum located far away from the building. Either levelling or hydraulic settlement gauges was used to measure the settlement.

b) Observation of the tilt of the building, if any. This was done by suspending plumb lines from convenient locations on the wall and measuring the offset of the external walls from the plumb line.

c) Development of cracks in walls/floors. Glass tell-tales were fixed across the cracks and crack widths were measured.

The purpose of above observations was to monitor the deformation / distress in the building and to decide upon the measures (temporary or permanent) to be taken to protect the building, if necessary. In general, temporary measures were taken whenever the rate of settlement appeared high and cracks became alarmingly wide. Providing salballah props below beams and floors and filling the cracks with cement mortar were generally resorted to.

Of a large number of buildings thus monitored during the metro construction about 60 buildings (Group A) had shown small and minor damages in the form of cracks in walls and floors and minor tilts. They had not shown any major structural distress and had not required any detailed observation other than monitoring the cracks and tilts.

Some buildings, however, suffered moderate to severe damages and temporary protective measures had to be taken from time to time pending final rectification at a later date. Twenty eight such buildings (Group B) were brought under this study - seven in the Southern section and twenty one in the Northern section.

For these buildings detailed settlement records were kept in addition to the observations of crack width, tilt etc.

Group A Buildings : An examination of the relevant data for the Group A buildings revealed that almost all the buildings were pretty old, being constructed between 40 and 100 years ago. They were of conventional load bearing wall construction, two to four storeys high. They were generally provided with shallow strip foundation. The buildings did not appear to have any regular maintenance. Peeling plasters, exposed brick work, damp walls were common sights. By the time the first signs of distress were noticed, the 'cut-and-cover' construction in the adjacent stretch had already progressed to some extent. Observations were made to see if and how these damages progressed with further excavation.

The buildings had suffered mostly architectural damages which had shown up as cracks in the walls and floors and minor tilt in the buildings. The cracks were mostly just hair line cracks although in some cases wider cracks had been noticed. The vertical tilt, as observed by measurements from the plumb line indicate very low values of angular distortion ranging from 1 in 600 to 1 in 1500 with an average of about 1/1000. Such order of tilt should not lead to any major distress in a conventional building. However, poor maintenance of the buildings might have resulted in progressive loss of strength of the binding mortars and caused damage. The distress to these buildings had generally been arrested after the subway box was cast and backfilling completed. By and large, they did not give rise to any long term problems.

Group B Buildings : These buildings had undergone relatively large movement during construction and consequently, they had suffered more distress.

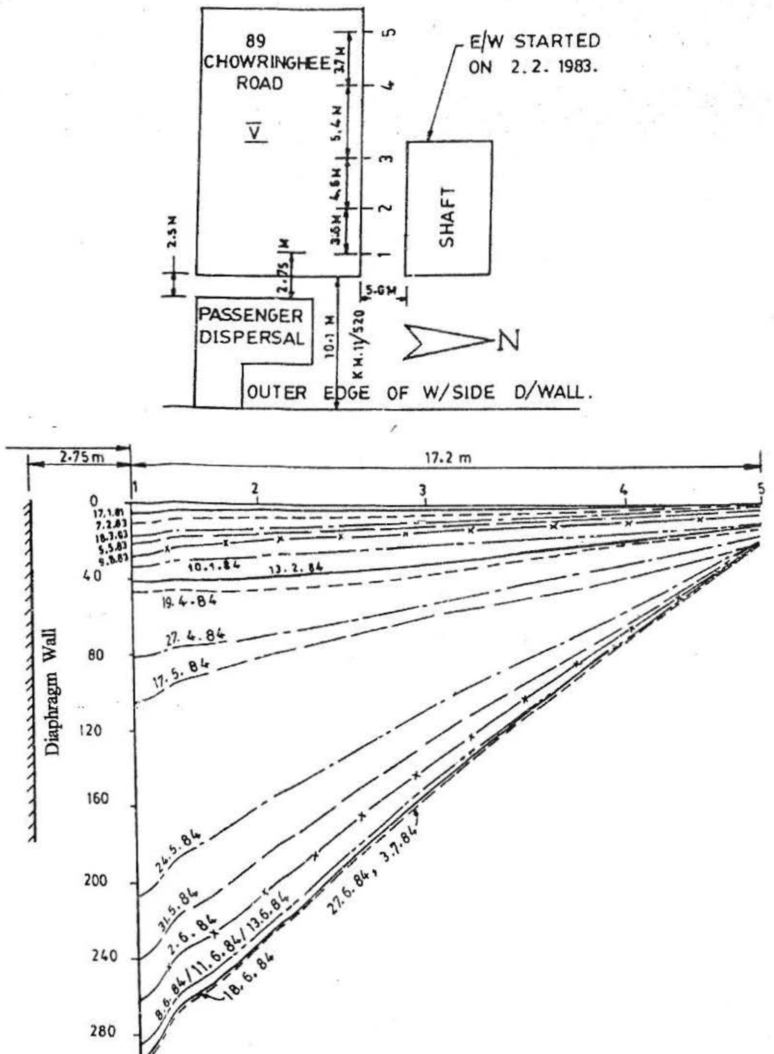


FIGURE 66 : Building Settlement due to Braced Cut – Premises No. 89, Chowringhe Road

Settlement observation of the plinth lines of these buildings normal to the alignment were made on different dates since distress was first noticed. Figs. 66 to 68 give the data for some of the buildings thus observed.

For most of the buildings the first set of observations was made when the excavation had already made some progress. Accordingly, the settlement data as recorded would not give the total settlement of these buildings. On the basis of the observations of ground settlement as indicated earlier an estimated settlement of 0.5% of the depth of the cut as existing on the date

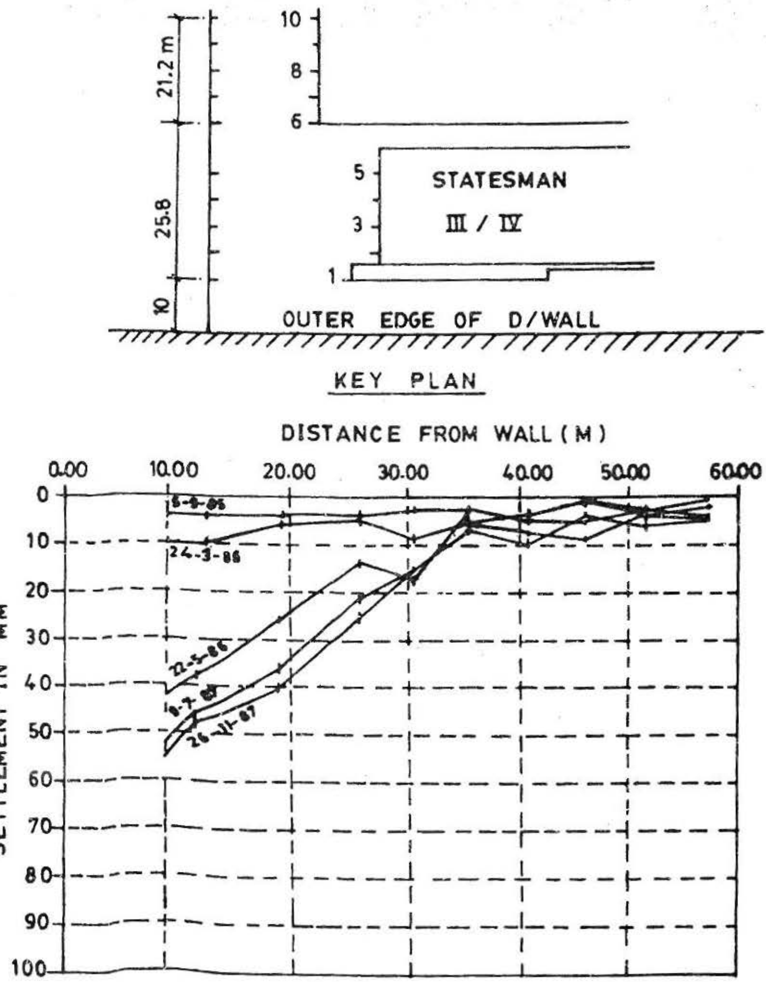


FIGURE 67 : Building Settlement due to Braced Cut – Statesman House

when observations were started was added to the observed settlement to obtain an estimate of the total settlement of the building. Taking this into account, the maximum settlement of different buildings has been obtained. The results are shown in Table 8.

The total settlement of the buildings in the Southern section, varied from 70 mm to 355 mm with excessive settlement in three buildings viz : 160E, 162 and 164, S.P. Mukherjee Road. The maximum settlement expressed as percentage of the depth of the cut ranged between 0.55% and 0.80%. These values are within the upper limit of ground settlement indicated earlier.

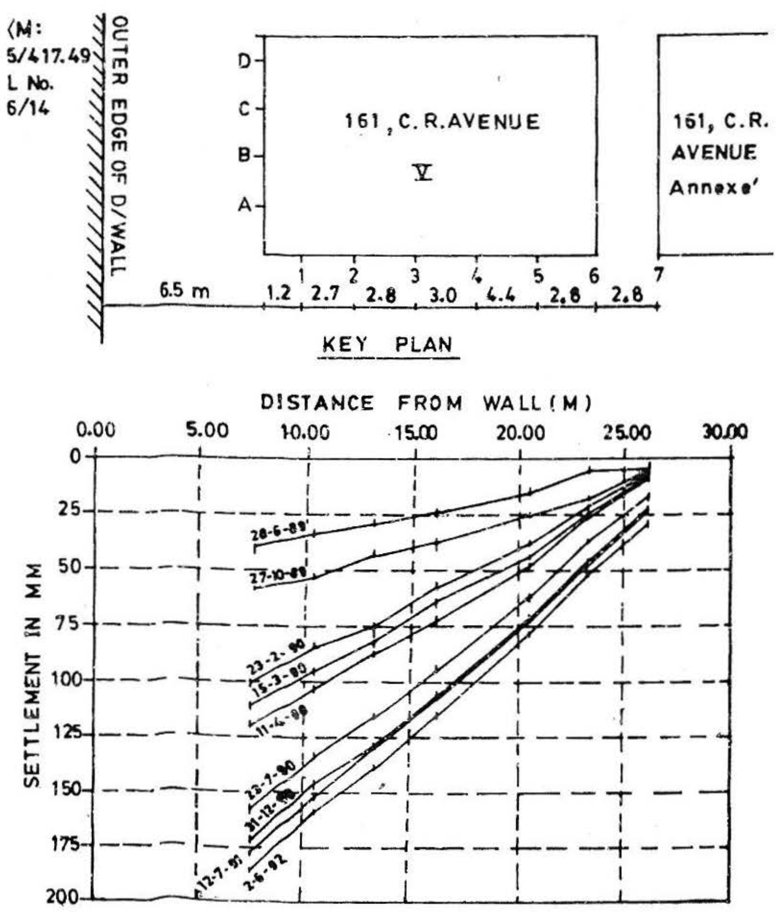


FIGURE 68 : Building Settlement due to Braced Cut – Premises No. 161, C.R. Avenue

Table 8 Maximum Settlement off Buildings

Premises No.	Section	Distance of building from outer edge of D/wall (mm)	Depth of cut (H) (m)	Total Settlement (∂) (mm)	∂/H (%)
Southern Section					
47 A.T.M. Road	13C	4.8	13.67	110	0.80
48A A.T.M. Road	14A	7.0	12.93	70	0.55
92 S.P.M. Road (N)	14C	5.0	13.10	82	0.63
92 S.P.M. Road (S)	14C	5.0	13.40	95	0.70
160 E S.P.M. Road	15B	7.0	13.22	294	2.22
162 E S.P.M. Road	15B	8.0	12.72	168	1.31
164 E S.P.M. Road	15B	4.0	12.72	355	2.75
Northern Section					
Statesman House	CS-9	10.0	12.0	72	0.60
Medical College	CS-8	3.8	14.0	136	0.97
Islamia Hospital	CS-8	1.8	14.5	214	1.48
Mahajati Sadan	CS-7	13.5	14.0	94	0.67
161 C.R. Avenue	CS-7	6.5	13.0	203	1.56
164 C.R. Avenue	CS-7	8.0	14.0	296	2.11
170 C.R. Avenue	CS-7	2.0	14.0	637	4.44
170A C.R. Avenue	CS-7	2.0	14.0	856	6.11
180A C.R. Avenue	CS-7	1.8	14.0	153	1.10
244 C.R. Avenue	CS-6B	4.0	14.0	392	2.80

However, three buildings in section 15B, including premises no.89 Chowringhee Road, Fig. 66, had undergone maximum settlement of 1.31% to 2.75% of the depth of cut. These are much in excess of the general pattern of ground settlement observed for most of the Calcutta Metro alignment. Careful examination of the excavation adjacent to these buildings revealed a clear gap between two adjacent panels in the diaphragm wall which was subsequently bridged by vertical steel laggings welded to the diaphragm wall reinforcements during excavation, Fig. 69. Plenty of water seeped through the laggings during excavation, obviously because the gap in the diaphragm wall below any level remained unbridged till further excavation was done. This had resulted in considerable soil loss which had to prevented by placing sand bags from time to time. This must have contributed to the large building settlement in this section. Also, subsequent construction of passenger dispersal facilities close to the building increased the settlement considerably.

The settlement of the buildings in the Northern section, varied from 72 mm to 856 mm. The Statesman House, showed typical settlement profile observed for most of the buildings in this section, Fig. 67. The magnitude of settlement, on its own, was not high but delay in fixing struts, after the appropriate excavation stage was reached, after caused additional settlement causing distress to some buildings.

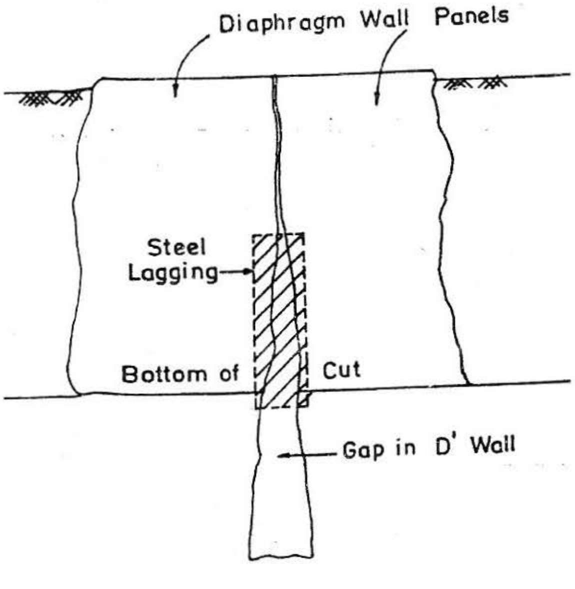


FIGURE 69 : Steel Laggings to Bridge Discontinuity in Diaphragm Wall

The buildings 170 & 170A C.R. Avenue had undergone excessive settlement of 637 mm and 856 mm respectively. Enquiries reveal that there was a halt in construction for almost a year due to a lockout in the construction agency and the trench was filled with water for about a year. During this time, the settlement at different points in the building had increased appreciably. After construction was recommenced and grouting in the surrounding soil was done, the settlement could be brought under control. Excessive settlement of some buildings in the region is understood to be mainly due to water seepage into the trench during the prolonged pause in construction.

Settlement at Difference Stages of Excavation

Table 9 gives a summary of the settlement data of some buildings at different stages of construction. The three stages of construction have been separated as follows :

a) Excavation stage

b) Box construction stage

c) Back filling stage

The time taken for each stage of construction mentioned above have also been indicated in Table 9. Some interesting observations may be made from these data :

a) By far the greatest part of the settlement occurs during the excavation stage. On average this settlement accounts for as much as 56% of the total settlement.

b) The settlement occurring in the period during which the subway box is constructed is also substantial - accounting for nearly 28% of the total.

c) The settlement during backfilling stage is generally small. On an average it does not exceed 16% of the total settlement.

Thus, the total settlement has, by and large, occurred fairly equally during the excavation stage (56%) and the post excavation stages (44%). The post excavation settlement includes the settlement during box construction and backfilling stages. Separating the two however, becomes difficult because the backfilling does not always follow the box construction immediately. Where backfilling is started late but completed fast the corresponding settlement should be low.

Table 9 Building Settlement at Different Stages of Excavation

Building	Section	Depth of cut (m)	Time to reach full cut depth (days)	Time for box construction after excavation (days)	Time for back filling (days)
1	13C	13.67	270	165	30
2	14A	12.93	280	75	10
3	14C	13.10	450	270	40
4	14C	13.10	230	150	20
5	15B	13.22	350	120	40
6	15B	12.72	340	200	40
7	15B	12.72	290	290	175*
8	CS8	14.5	257	527*	—
9	CS7	14.0	252	188	90
10	CS6B	14.0	97	580*	196*
11	CS9	12.0	931*	565*	—
		Average	280	180	40

Table 9 Contd...

Building No.	Settlement during excavation ∂_1 (mm)	Settlement during box construction ∂_2 (mm)	Settlement during back filling ∂_3 (mm)	Total Settlement $\partial = \partial_1 + \partial_2 + \partial_3$	$\frac{\partial_1}{\partial}$ (%)	$\frac{\partial_2}{\partial}$ (%)	$\frac{\partial_3}{\partial}$ (%)
1	60	30	20	110	55	27	18
2	45	10	15	70	65	14	21
3	37	40	25	102	35	40	25
4	70	15	10	95	73	16	11
5	220	40	34	294	75	14	11
6	102	40	26	168	61	24	15
7	135	175*	45	355*	37	50	13
8	129	83	40	252	51	33	16
9	77	31	45	153	50	20	30
10	72	41	19	132	55	31	14
10.1	236	135*	21	392*	60	35	05
Average	105	37	28	170	56	28	16

[* Excessive high values not considered for averaging]

Table 10 Angular Distortion of Buildings

Premises No.	Section	Distance of building from D/wall	Depth of cut	Total Settlement	Angular Distortion	Zone of influence
Southern Section						
47 A.T.M. Road	13C	4.8	13.67	110	1/250	30
48A A.T.M. Road	14A	7.0	12.83	70	1.300	37
92 S.P.M. Road (N)	14C	5.0	13.10	82	1/300	36
92 S.P.M. Road (S)	14C	5.0	13.10	95	1/400	35
160 E S.P.M. Road	15B	7.0	13.22	294	1/45	30
162 E S.P.M. Road	15B	8.0	12.72	168	1/120	26
164 E S.P.M. Road	15B	4.0	12.72	355	1/58	30
Northern Section						
Statesman	CS-9	10.0	12.0	72	1/475	57
Medical College	CS-8	3.8	14.0	136	1/290	30
Islamia Hospital	CS-8	1.8	14.5	214	1/125	25
161 C.R. Avenue	CS-7	6.5	13.0	203	1/125	-
170 C.R. Avenue	CS-7	2.0	14.0	637	1/36	22
180A C.R.Avenue	CS-7	1.8	14.0	153	1/140	14
230 C.R. Avenue	CS-7	4.0	14.0	153	1/202	18
244 C.R. Avenue	CS-6B	4.0	14.0	296	1/75	30

Angular Distortion

From the settlement profile of the buildings it is possible to obtain the maximum angular distortion (∂/L) of the building at a particular stage of construction. Table 10 summarises the angular distortion of the buildings at significant stages of construction. The ∂/L values increase generally with increasing maximum settlement. Fig. 70 shows a plot of maximum settlement vs. angular distortion for the buildings. It can be seen from that ∂/L increases appreciably for maximum settlement greater than 100 mm. Below ∂ = 100 mm ∂/L is generally less than 1/300. Thereafter ∂/L increases to as much as 1/40 for maximum settlement of the order of 300 mm. It is to be kept in mind, however, that the ∂/L values indicated in Table 10 do not give the absolute ∂/L for the buildings under consideration. They only reflect the angular distortion occurring during the construction of the Metro. These ∂/L values are in addition to the angular distortion that would have occurred prior to the start of Calcutta Metro construction. Most of these building had been built a long time ago and they must have undergone some differential settlement already. Generally, a ∂/L of 1/300 is considered to be the limiting value of angular distortion beyond which a conventional load bearing wall construction is likely to show some distress.

First, architectural damages in the form of cracks in walls/floors are likely to appear. With further angular distortion more severe distresses would occur.

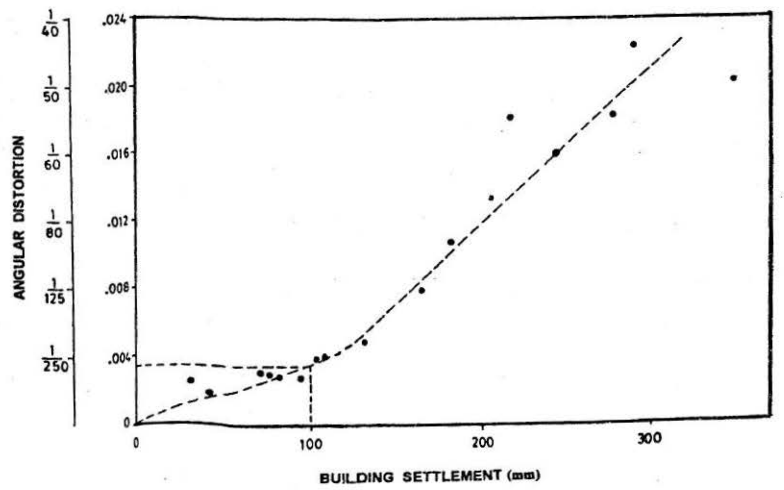


FIGURE 70 : Maximum Settlement Vs. Angular Distortion of Building

Placement of Strut

Braced excavation in soft clay is normally done with struts placed at intervals of 3-4 m vertically. A strut may, however, be placed only when the excavation has gone slightly below the desired strut level. When the excavation reaches the strut level the strut is supposed to be placed immediately before further excavation is done. If the fabrication of the strut is delayed the contractor, to save his labour going idle, tends to carry on with the excavation and by the time the strut gets ready for installation an additional 1-2m may have been excavated. Looking at the struts afterwards does not show any deviation from design. Only a critical appraisal of the time and sequence of activities would reveal that the unsupported cut depth had been more than what was provided for in design. In order to study the effect of strut spacing the settlement that occurred for the excavation between first and second strut installation for different test sections are plotted in Fig. 21. Settlement values are taken for the total excavation done below first strut level prior to placing the second strut including the depth of over cut, if any. It may be seen that the settlement increases almost linearly for unsupported cut depth of upto 4 m. Thereafter the settlement increases rapidly with depth. Accordingly, a strut spacing of 3 - 4 m was adopted for design. But often there is overcut prior to placing the second strut. This leads to unsupported cut depth of more than 4 m.

The resulting additional ground settlement has often led to damage to adjoining structures which could have been prevented had the struts been placed in time and as per the desired construction sequence.

Time Effect

In order to ascertain the reasons for large settlement of some buildings along the alignment it is necessary to consider the effect time on settlement. Table 9 gives the time required for different stages of construction in the relevant sections. The excavation period has generally varied form 200-300 days except for some buildings where unusually long periods of time were required to do the excavation. Settlement during this period varied between 37 mm to 236 mm. Similarly, the time taken between reaching the excavation depth and casting the subway box varied widely (75 to 290 days). During this stage the settlement varied from 10 mm to 40 mm. Three buildings, however, took very long time for box construction (527 to 580 days) due to stoppages of work. This inevitably led to higher settlement (83 to 135 mm).

Figure 71 shows the effect of time required for box construction or due to any pause in construction on the settlement of adjacent buildings. Settlement during this period (S_t) has been expressed as a ratio of the excavation stage settlement (S_o). It will be seen that the ratio (S_t/S_o)

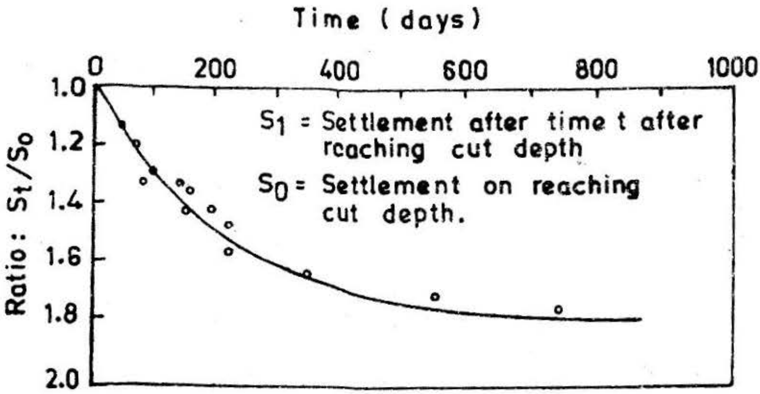


FIGURE 71 : Settlement of Adjacent Building – Effect of Time of Construction

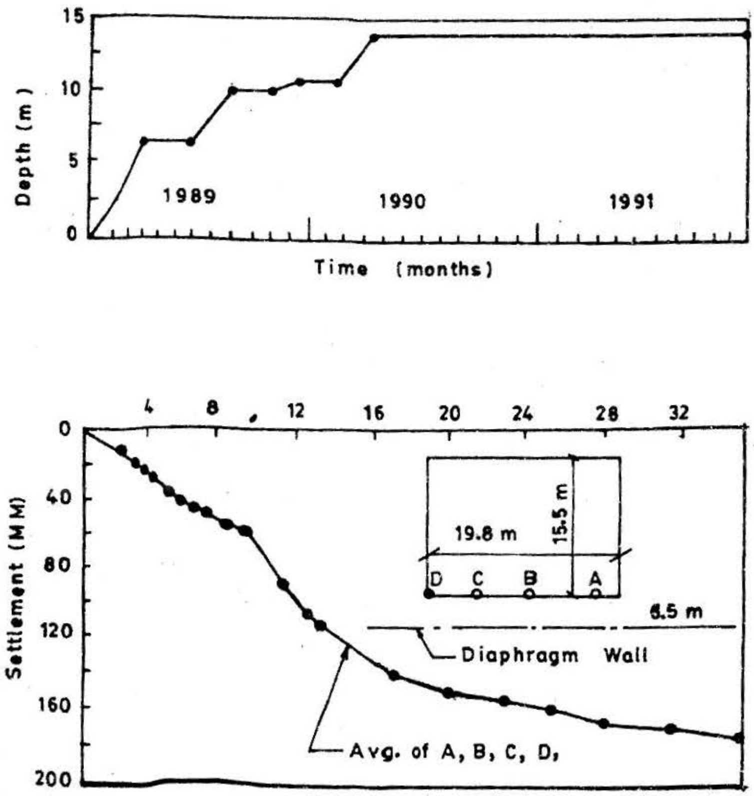


FIGURE 72 : Settlement of Premises No. 161 C.R. Avenue – Effect of Time

increases rapidly for pause in construction upto 150 days during which additional settlement of upto 40% has occured. Thereafter, the rate of increase of S_t/S_o goes down sharply until the curve becomes asymptotic to the value of S_t/S_o approximately equal to 1.8.

It appears from the data that, if the box construction is done in 100 days, further settlement of only 30% of the excavation stage settlement occurs while the settlement goes upto 60 % of the excavation stage settlement if the box construction takes 250 days.

It seems the excavation for Calcutta metro construction has taken as average time of 260 days, leaving out the extreme case of building 11, during which settlement of 105 mm has taken place for the typical 12-14 m deep cuts. If it is considered that this time is just about the optimum that can be achieved in the field a settlement of the order of 100-110 mm may be expected during this period. Further settlement can be restricted appreciably if the time for box construction can be controlled. For an excavation stage settlement of 100 mm additional settlement for different periods of box construction may be expected to be as follows:

Excavation stage settlement (260 days)	:	100 mm
Additional settlement during box construction, if done in :		
100 days	:	30 mm
150 days	:	40 mm
200 days	:	50 mm
250 days	:	60 mm

The data emphasize the need for speedy construction of underground facilities once the bottom of excavation is reached. If the optimum period of box construction is taken as 200 days the settlement during this period will not exceed 50% of the excavation stage settlement. Slower progress is likely to result in larger building settlement.

The effect of time on ground settlement may be considered in further detail. Often the excavation is left open for long periods of time at the final or any intermediate cut level thereby causing additional settlement at that level. For each test section where the excavation has been kept open at any level for long periods of time the maximum settlement adjacent to the cut after the particular depth was reached has been measured. Further settlement with the excavation remaining at the same level has also been measured against time. The data for one such building (161 C.R. Avenue) are shown in Figs. 68 and 72. It may be noted that, for this building, the excavation was kept open at 8 m depth for 90 days. Thereafter, excavation was resumed and the final cut depth (14m) was reached in another 85 days on 15.03.90. Lean concrete was laid on 15.04.90 but after that there was a long hold-up, because of labour dispute in the construction firm. Work was resumed only on 30.01.92 when the base raft of the subway box was cast. Admittedly this is an extreme case where work was stalled for unduly long period of time. But the settlement data for the buildings clearly show the effect of time on ground settlement adjacent to the cut.

It will be noticed that out of total settlement of 172 mm no less than 93 mm has occurred during the stoppages of excavation. True, the sequence of placing struts and over cutting at any depth prior to installation of struts may also have some effect on the additional settlements that are seen to have occured. But the pattern of settlement as shown in Fig. 72 clearly shows the predominant effect of time on settlement of the building. Similar data for other buildings affected by prolonged construction illustrate the effect of ageing and creep on the deformation of soil in braced excavation in soft clay.

Conclusion

The paper presents the case history of the geotechnical aspects of Calcutta Metro construction. A 16.5 km long underground railway was built for the first time in India for the Calcutta rapid transit system. The subway box was built mainly through the silty clay of normal Calcutta deposit about 12-15 m below G.L. by cut and cover method. The construction was done mostly with RCC diaphragm walls propped against each other by steel struts. Detailed soil investigation was done to obtain the soil parameters for the design and construction of underground works. The design of the braced cut was done to ensure stability against bottom heave and clay bursting while the rigidity of the support system was ensured by providing steel struts at 3-4 m vertical and longitudinal spacing.

The performance of the braced cut was studied by extensive field measurements including detailed instrumentation at three test sections. A theoretical analysis has been done to develop a mathematical model for performance evaluation of the braced cut and to make a parametric study of the influence of various factors. The relevant construction control parameters are identified and the effect of braced cut on adjoining structures are studied from field survey and measurements.

The paper presents the design methods adopted for the metro excavation and gives recommendations for depth of diaphragm wall and spacing of struts in such construction. From bottom heave consideration it is sufficient to take the diaphragm wall to a stiff layer below the cut. The parametric study identifies strut spacing, width of cut and depth of diaphragm wall as the major factors that influence the ground movement adjacent to the cut. The instrumentation data provide insight into the development of earth pressure on the diaphragm wall, strut load and ground movement. The importance of construction control measures for such underground construction is highlighted. Deviations from design, delay in strut installation, time effect etc. have major effect on the performance of braced cuts and the safety of adjoining structures. With proper control of excavation sequence and by ensuring rapid construction it should be possible to do underground construction in built up urban centres without any undue hazards.

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