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THE THEORETICAL AND SITE BEHAVIOUR OF A BRACED DIAPHRAGM WALL-A COMPARISON

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ABSTRACT

Three numerical analysis carried out for the design of a diaphragm wall were examined by the results of field observation data.

Utilizing the wall stiffness, supporting system and construction sequence, the relative merits of those factors on the analysis of diaphragm wall have been investigated and their effects are compared with the observed behaviour of the wall.

The predicted bending moment and wall displacement by elasto-plastic method agreed well with the observed values. The rigid slab supported system (i.e Top-Down Method) found to be the most effective way of controlling ground movement.

INTRODUCTION

A diaphragm wall or Slurry wall is an underground reinforced - concrete wall which has been extensively applied to the construction of underground space world wide since its first appearance in the early 1950s.

This relatively modern technique of deep basements construction method provide considerable merits in built up areas where other techniques may have limitations such as strict ground movement and environmental factors.

The behaviour of a braced diaphragm wall in practice is considerably influenced by soil parameters, wall stiffness, type of bracing systems, depth and size of excavation and construction sequence etc. In this paper all the important influencing factors have been investigated and their results compared with the field observation data.

Theoretical prediction of the wall performance was closely examined with evaluation of field data. Predicting the accurate design load for the wall is extremely difficult because of the magnitude and distribution of horizontal earth pressure are based on the relationship between wall displacement and mobilization of particular soil stress (i.e soil-wall interaction) which varies throughout the construction period.

Classical limit state method of Rankine - Coulomb theories and empirical equations suggested by Peck were reviewed and comparison design carried out for applicability of those methods to a rigid diaphragm wall.

FIELD DATA

The actual behaviour of a diaphragm wall can only be observed through fieled measurments or laboratory model tests.

The site chosen for the study is an area approximately 56m x 46m and varied in depth from 17.0m to 25.0m below ground level. A diaphragm wall 800mm thick was constructed into a dense sand layer to the depth of 28m. During the excavation the diaphragm wall is supported by steel struts and corner b- -s.

The ground soils consist of highly stratified layers of soft clay, sandy silt, sandy gravel and dense sand with low level of ground water table. Made ground surface of 2 - 3m thick is followed by soft clay to a depth of 10m overlying the more than 20m thick sand and gravel deposit as shown in Fig 1.

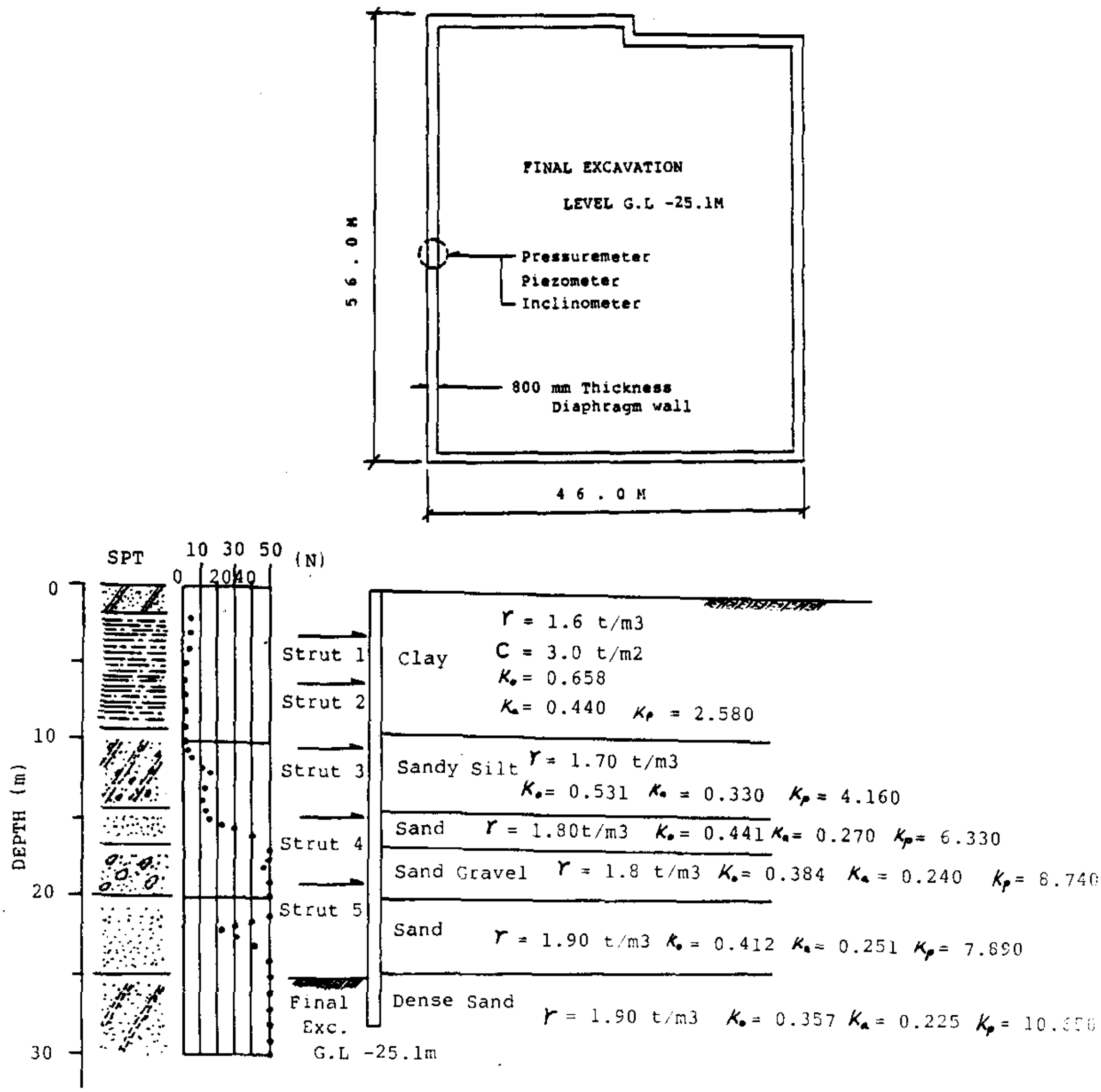


Fig 1 Site plan and Soil profile

The field data from the instrumentation and monitoring system was provided by the OHBAYASHI Corp. who has considerable experience in this sphere.

In order to ensure the stability and safety of the diaphragm wall for the deep excavation on this construction site, the following instrumentation system was installed.

- Earth and water pressure (Pressuremeter and Piezometer)
- Stresses and strains in the diaphragm wall (Strain gauge and Inclinometer)
- Struts Forces (Pressuremeter and strain gauge)
- Surrounding ground settlements

Location of instrumentation and the details are shown in Fig 2.

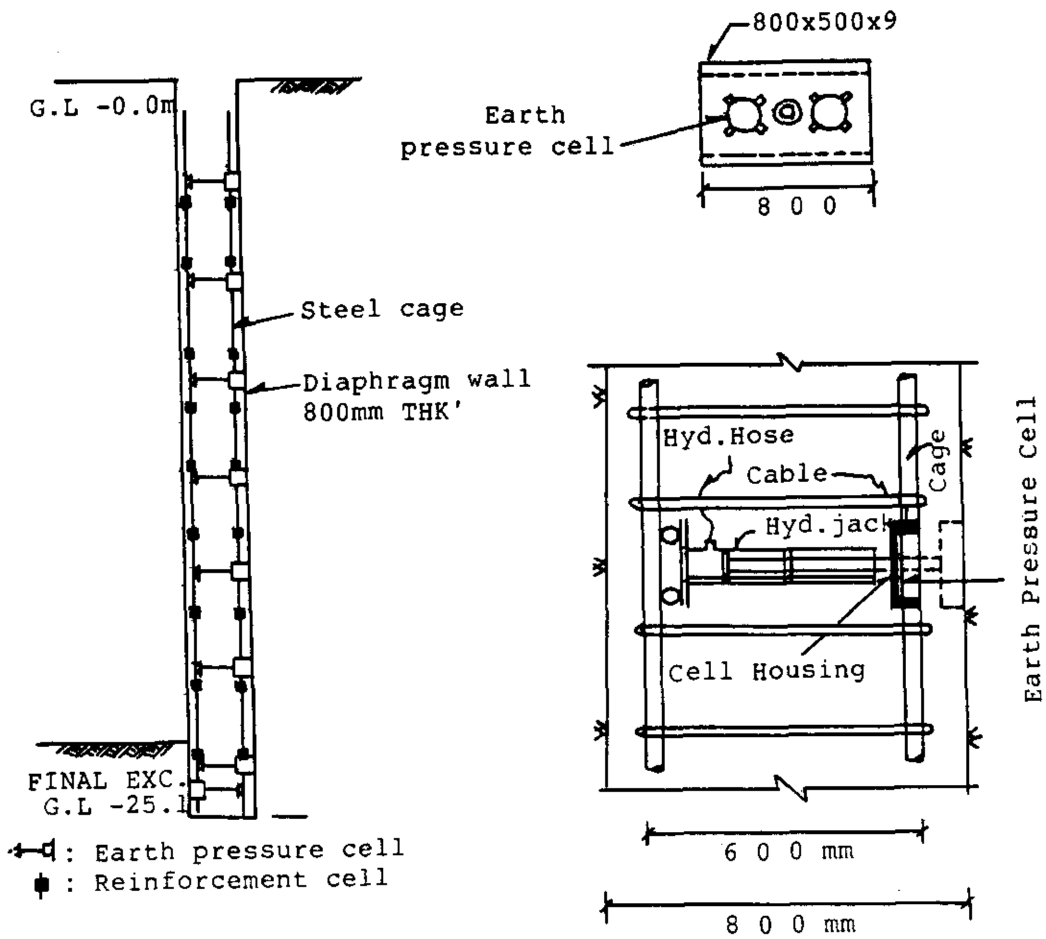


Fig 2 Location of instrumentation and the details

In monitoring the diaphragm wall system, particular difficulties may be experienced when all the instrumentations are to be fixed to the steel cage so that hydraulic jack can push the earth pressure measuring units firmly toward the excavated soil faces prior to concreting operation.

ANALYTICAL METHODS

The field behaviour of a diaphragm wall was examined by various analytical methods which are frequently used in diaphragm wall design practice.

Three analytical design methods to predict diaphragm wall behaviour are being tested by the results of the field data.

DESIGN METHOD(1) - Elastic Analysis

The force, moment and displacement of the diaphragm are expressed to a set of independent homogenious equations in matrix form and solved by Gaussian elimination technique.

General procedure of this widely available plane frame program

```
                         Start
                           |
   For all elements   [ Form element, stiffness matric [ K* ] ]
                      [ Assemble Global stiffness matric [ K ] from [ K* ]'s ]
                      [ Apply boundary conditions and loads ]
                           |
                      [ Solve for { δ } and reactions ]
                           |
   For all elements   [ Evaluate member forces ]
                           |
                         Stop
```

INPUT DATA :

1) RANKINE EARTH PRESSURE

2) PECK's EMPIRICAL EARTH PRESSURE

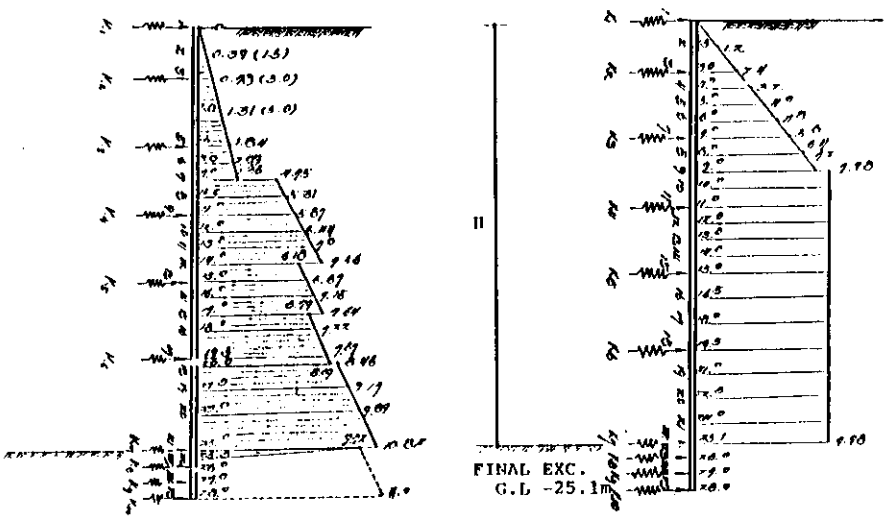


Fig 3 Spring Model 1) and 2) of theoretical models.

1) Rankine earth Pressure : $P_a = \gamma H K_a + q K_a - 2 C \sqrt{K_a}$

2) Peck's empirical pressure : CLAY : $P_a = 0.5 \gamma h$

SAND : $P_a = 0.65 \gamma H K_a$

For temporary strut stiffness (H - 300 x 300 x 10 x 15)

Strut spring stiffness ;

$$K_s = \frac{EA}{LS} = 2734 \ t/m^2/m \ \text{Run} \quad \text{(Single)}$$

$$K_s = \frac{E_s}{1.35B} h B$$

$$K_h = \frac{E_s}{1.35B} \qquad \text{(Terzaghi, 1955)}$$

Soil spring stiffness : $K_s = K_h \ h \ B$

where K_h = Coeff. of horiz. subgrade reaction

B = Width of element (here = 1m)

h = Spring spacing

E_s = Young's modulus of soil

L = half total length of site

S = spacing

DESIGN METHOD(II) - Modified Elastic Analysis

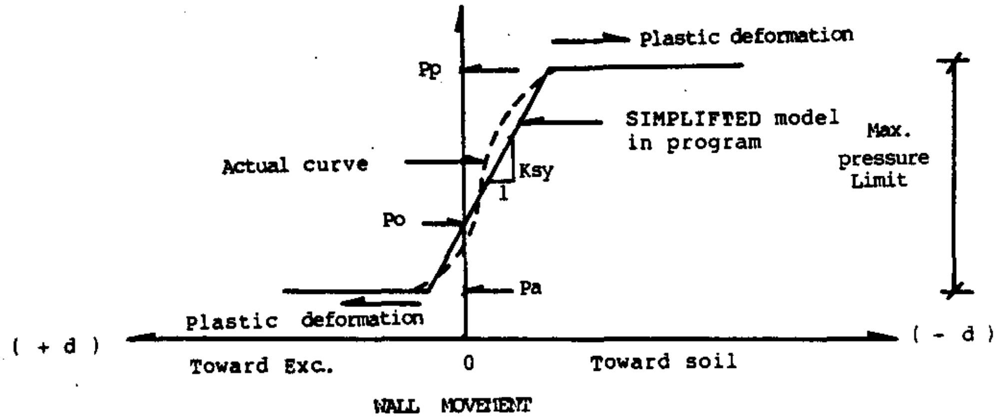
The mathematical computation is similar to the design method(I) and the following three major points have been considered.

- $K_s = 40 \times (1.3 N_c + P_v N_q + 0.5 \gamma B N_r)$ as soil spring stiffness
- Whichever greater earth pressure diagram between Rankine and Peck's is selected for the analysis.
- The allowable angular rotation of diaphragm wall is limited to 1/400.

DESIGN METHOD (III) - Elasto-Plastic Analysis

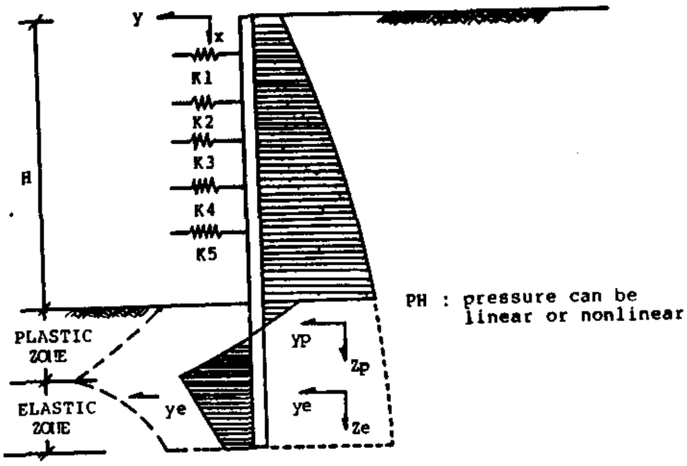
Elasto-Plastic method of diaphragm wall analysis is often known to represent the most realistic behaviour in the field. All the results are generated by a WALLAP Program which was modified to suit for reinforced concrete diaphragm walling, and an adapted finit element method in which the wall and soil are modeled as a beam and springs. (Fig 5.)

Soil is assumed to have passive and active limits of earth pressures by Rankine equations. The earth pressure should be within this limits, therefor the stresses within these limits are considered to be linear elastic. (Fig 4.)



$$\int (K_a \sigma_v' - 2C\sqrt{K_a}) dz \leq \int P\, dz \leq \int (K_p \sigma_v' + 2C\sqrt{K_p}) dz$$

Simplified elasto-plastic analytical model (Fig 4 above and Fig 5 below)



Ⓗ Area $EI \dfrac{d^4y}{dx^4} = ax^2+bx+c$

Plastic Zone $EI \dfrac{d^4y_p}{dz_p^4} = -(\gamma_t K_a Z_p + 2C\sqrt{K_a} - P_H)$

Elastic Zone $EI \dfrac{d^4y_e}{dz_e^4} = -E_s y_e$

A brief description of the computer program is as follow;

- Dividing diaphragm wall into finite elements.
- Formation of flexural rigidity equations for each element in term of M, V, θ, δ.
- Assemble a global stiffness matric for the wall.
- Boundary conditions and supporting systems.
- Soil stiffness matric for stress(pressure) and wall deformation($\delta = K\ P$)
- Assemble the wall and soil stiffness matrics to allow for the interaction between earth pressure and wall movement.
- Evaluation of bending moment, shear force, wall deflection, and support reaction by solving the matrics.

This program accumulates wall deflection from previous stages of construction and prestressing of the wall is also taken into account in term of reacting pressure (load).

BENDING MOMENT

Field observed bending moments at various construction stages are compared with computer outputs of the elastic design method and elasto-plastic analytical method as shown in Fig 6.

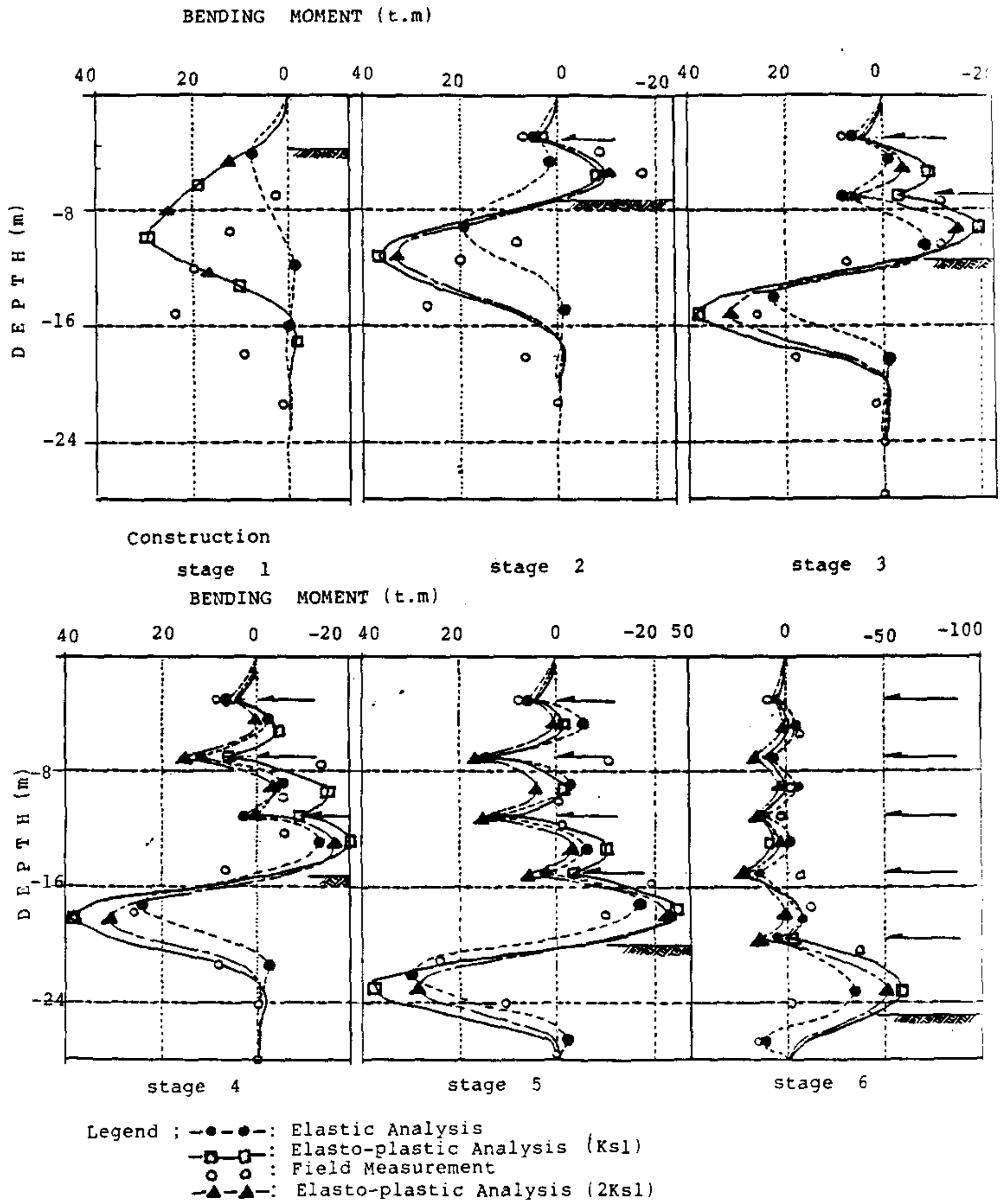


Fig 6. Comparison of measured bending moment of strutted diaphragm wall with various numerical methods

The field data was obtained from 18 strain gauges attached to the reinforcement cage in the diaphragm wall panels.

The calculated moments from elasto-plastic analysis for the construction stage 1 at - 3.5m exc. and stage 3 at - 7.5m exc. closely approximate the field measured values, but some deviation occured in the elastic design method. Both calculated bending moments in the 5th stages through 11th construction stages are generally in good agreement with the field measured values.

It may be concluded from the results that the elasto-plastic method of diaphragm analysis suggests reliable estimates for the wall behaviour so far as rigid wall displacement and bending moment are concerned, whereas the elastic design method yields slightly unstable values at same construction stage compared with the field observed data.

WALL DISPLACEMENT

Fig 7. demonstrates that deflected wall profiles for the numerical analysis and field observed data at various stages of construction.

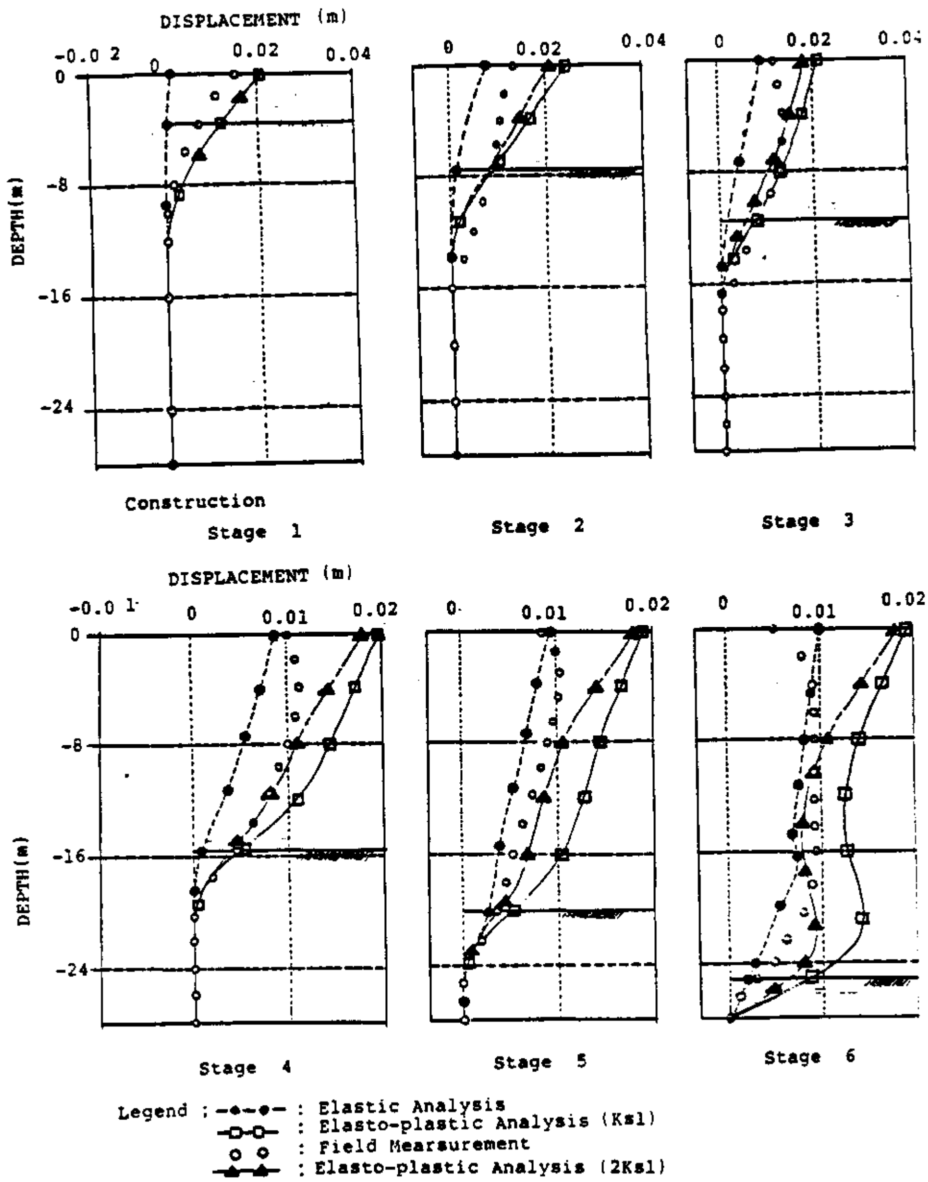


Fig 7 Comparison of measured Deflection of strutted Diaphragm wall using various numerical methods

Two analytical methods, elastic design and elasto-plastic design, with different strut stiffness were employed to investigate the actual behaviour of diaphragm wall on site.

From the plotted shape, the elasto-plastic analysis using strut stiffness of K_t = 4 E·A / S·L seems to produce the most satisfactory results referring to the field data, the wall displacement being intermediate between the observed and results of K_t = 2 E·A / S·L. This implies that the stiffness of struts may cause significant change in the wall deflection. The deflection values for the modified elastic analysis were noticeably smaller than the observed data.

This difference could probably be attributed to the computation method in which only a particular construction stage is analyzed independently at a time. Whereas the elasto-plastic method of analysis take into account all the deflected values of previous stages therefor providing greater magnitude of wall horizontal movement.

The reasons for the considerable deflection occured in the elasto-plastic analytical model would probably be the modelling of the real soil behaviour and acknowledement of construction sequence in the program.

EARTH PRESSURES

Horizontal earth pressure acting on the diaphragm wall are given in Fig 8 for various analytical predictions and measured field data. Here the earth pressure evaluated from the elasto-plastic model and classical theories of Rankine and Coulomb are compared with the observed values.

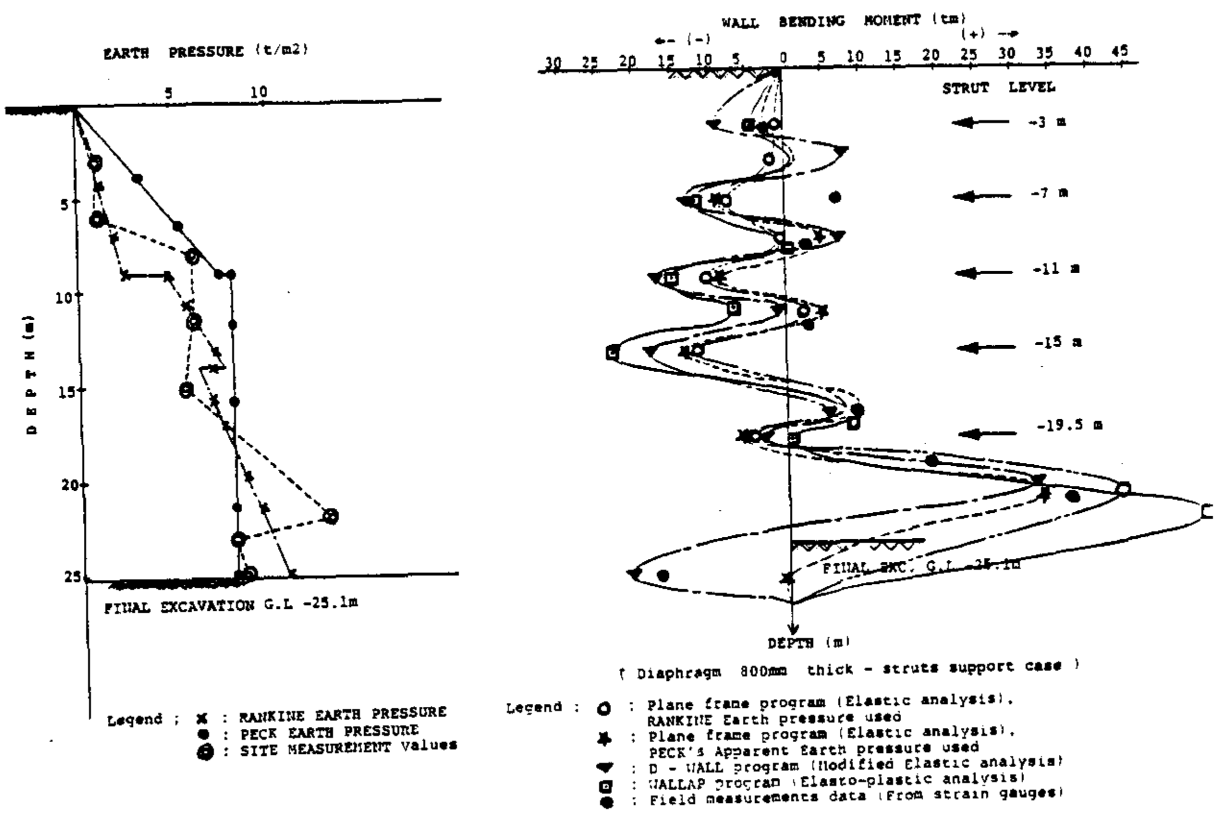


Fig 8. Horizontal earth pressure and bending moment for varios theories

In the upper clay layer, Rankine theory followed very closely with the observed data whereas other analytical methods showed relatively conservative values over the entire clay layer. In the mid. part of the wall, the observed data retained well within the analytical estimations employed. Near the toe of the wall, an irregular pressure pattern was produced on the observed curve. Rankine and elasto-plastic methods are likely to be fairly reliable for this particular location whereas the predicted values by the Peck's empirical equation are rather small.

It may be concluded from the above results that the earth pressure shape for the diaphragm wall followed neither classical Rankine theory for rigid wall nor empirical Peck's trapezoidal pressure distribution, but it seems to lay in between the two methods or rather close to the classical triangular shape of pressure distribution

An accurate estimation of lateral earth pressure around the diaphragm wall is almost impossible because of the complexity involed in the following major dynamic factors

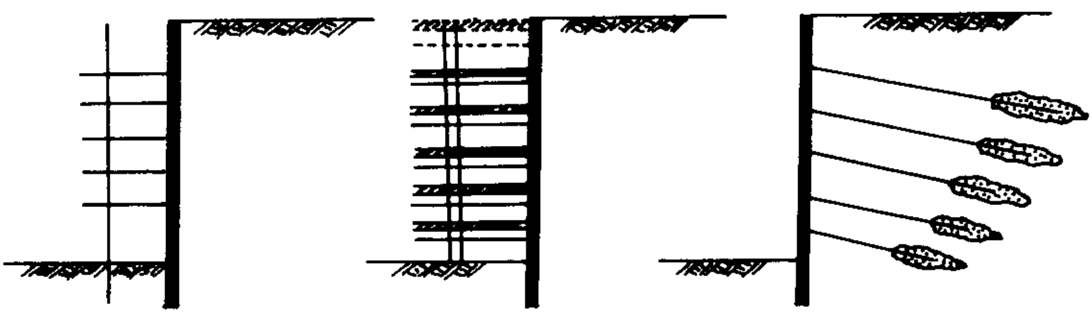
- Method of wall installation (in-site Construction system may reduce earth pressure at rest.)
- Construction sequnce and duration of excavation
- Wall property, supporting system
- Homogenity and isotropic state of soil conditions
- Embeded depth of the wall below the final excavation level
- Type of surcharge load and pre-load application
- Movement of ground water table
- Soil-wall interaction during construction stages (i.e number of multi-supporting levels)

The elasto-plastic method of diaphragm wall design a gives good estimate of design load(lateral pressure) by regarding the observed data.

SUPPORTING SYSTEM

Effects of supporting systems on the behaviour of diaphragm wall are shown in table 1.

Table 1. Effect of supporting system on the behaviour of diaphragm Wall (values for the final excavation stage only)



<STRUT ①,②> <TOP-DOWN ①,②> <EARTH ANCHOR>

① $KS1=\frac{2EA}{S.L}$ ① Basement Slab from -3.0m level Preload of 80 ton

② $KS2=2KS1$ ② Construct ground floor slab

	STRUT (Steel)		TOP-DOWN Ground floor slab		EARTH ANCHOR
	① KS1	② 2KS1	without	with	
Maximum bending momount (t·m)	58	50.9	45.1	47.3	47.3
Maximum horizontal displacement (mm)	19	18	18	7	16
Mean horizontal displacement (mm)	14.5	10.6	8.4	4.6	8.5
Maximum surface settlement (mm)	40	30	24	12	23
Strut Forces(Tons)					
Level 1	10.08	9.31	8.52	14.64	11.58
2	26.36	28.74	30.70	25.26	28.42
3	25.67	24.76	24.52	24.25	27.87
4	36.02	33.91	32.23	33.05	46.35
5	44.33	48.83	53.16	53.92	46.86
Σ	142.46	145.55	149.13	151.12	161.08

The 800mm thick diaphragm wall was subjected to various type of supporting systems, such as strut, basement slab and earth anchor, in order to examin influence of the supporting systems on the wall behaviour.

It can be seen from the table that the maximum bending moment in the less stiff strut system generated 28% more than those for the stiffer basement slab system in the top-down case. As the results of increasing the struts stiffness double on this site, the maximum bending moment and mean horizontal displacement of the wall have been reduced to 14% and 37% respectively.

Mean horizontal displacement was calculated from the volume of deflected wall shape divided by the wall height and it can be used for the computation of ground surface settlement behind the wall.

Top-down method reduced the maximum bending moment and mean horizontal displacement by 23% - 28% and 73% - 315% respectively. The preloaded of 80ton earth anchor system was as effective as Top-down method. Base on the analytical results of this particular project, Top-down method was found to be the most satisfactory so far as the movement of wall and ground settlement behind the wall are concerned

The earth pressure distribution diagram, bending moments and the wall displacement plot for the final excavation stage are shown in Fig 9 and 10.

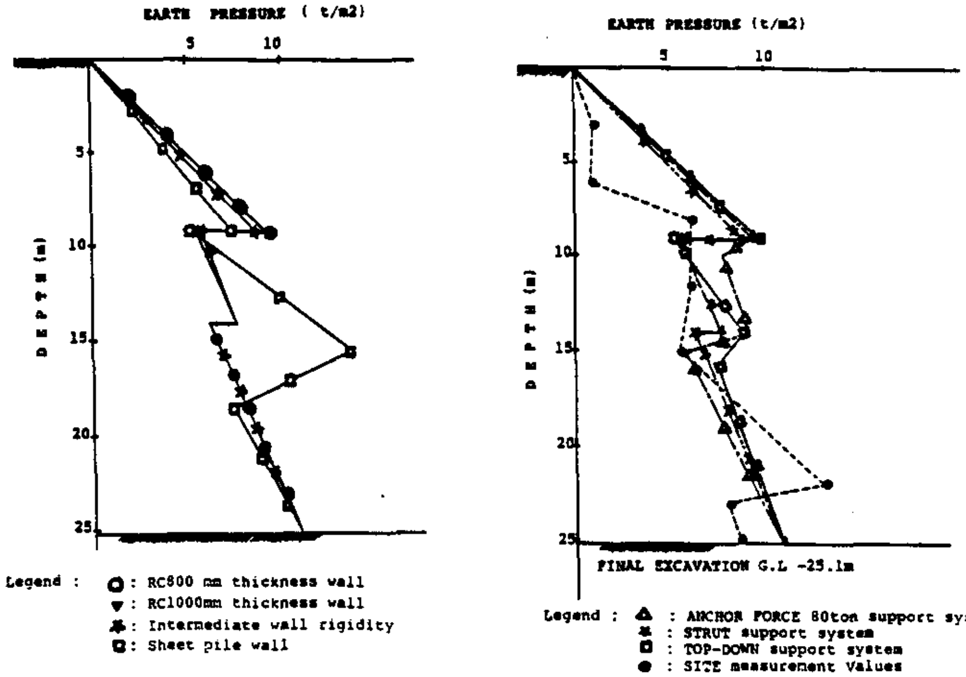


Fig 9 Earth pressure distributions for various wall rigiditys and supporting system

It is intersting to note that the maximum earth pressure and negative bending moment at the mid. height of the wall occured in the earth anchor supporting case. This phenomenon may be closely related with the arching and redistribution of pressure on the anchored flexible wall.

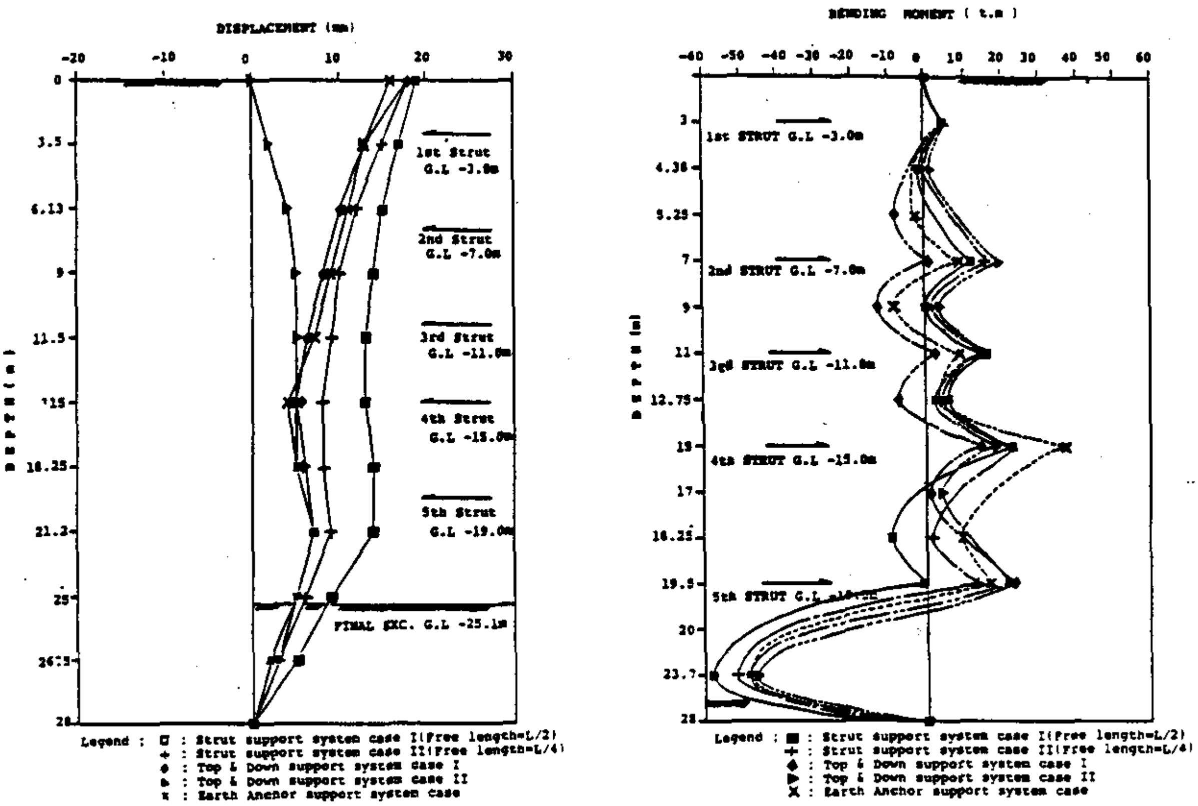


Fig 10 Bending moments and wall displacement diagram for various supporting systems

RIGIDITY OF WALLS

Effects of wall rigidity on the behaviour of strutted earth retaining walls are given in table 2.

Table 2 Effect of wall rigidity on behaviour of strutted earth retaining structures (Final execavation stage only)

	TYPE	E(t/m^2)	I(m^4)	A(m^2)
Diaphragm wall	THK 1000mm	$2.0x10^6$	$8.33x10^{-2}$	1.0
	THK 800mm	$2.0x10^6$	$4.27x10^{-2}$	0.8
Intermidate wall	$\frac{D.W+SHEET}{2}$	$1.2x10^7$	$2.15x10^{-2}$	4×10^{-1}
Sheet pile wall	ZP - 38	$2.1x10^7$	$3.83x10^{-4}$	9.4×10^{-3}

	Diaphragm Wall		Intermediate $\frac{D.W + Sheet}{2}$	Sheet Pile
	THK 1000mm	THK 800mm		
Maximum (T.M) bending moment	68.5	58.0	50.5	43
Maximum (mm) horizontal disp.	15	19	31.5	44
Mean horizontal disp. (mm)	12.5	14.5	20.5	26.5
Max. surface settlement (mm)	34	40	59	78
Strut Forces Level 1 (Tons)	10.87	10.08	8.79	7.5
2	25.61	26.36	25.14	23.91
3	27.98	25.67	25.62	25.57
4	36.91	36.02	36.48	36.94
5	38.77	44.33	52.72	61.11
Σ	140.14	142.46	148.75	155.03

The maximum bending moment of the diaphragm wall is 35% more than that of sheet pile wall implying that the stiffer the wall the bigger the moment: The sheet pile wall showed 83% more wall displacement than the stiff wall in the same conditions.

Fig 9-1 shows earth pressure distribution, bending moment and wall displacement for the walls with various stiffness.

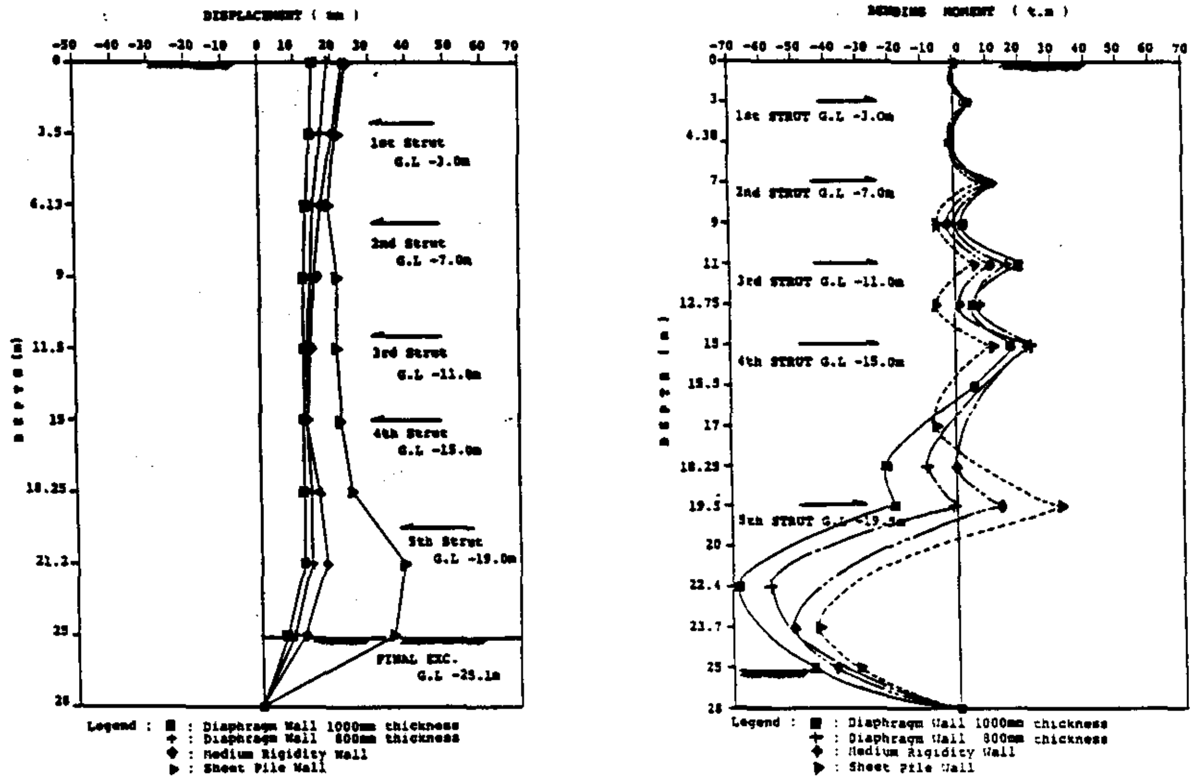


Fig 11 Bending moment and wall displacement for the walls with various stiffness.

The fig. 9 shows that the sheet pile wall near the toe of the wall deflection and negative bending moment greatly exceed those computed for other rigid type walls. Also a dramatic jump of earth pressure at the fourth level of prop is another surprising point of the sheet pile behaviour.

SURROUNDING GROUND SETTLEMENT

The ground settlement behind the wall is estimated using the approach set out by Caspe method 1966.

The maximum surface settlement for various diaphragm wall supporting systems, as well as the wall types, are summarized in tables 1 and 2. The results show a significant improvement can be achieved when the struts supporting system is replaced by Top-down method or preloaded earth anchors. There is a 67% - 330% reduction of surface settlement by employing the Top-down method and a 74% reduction using an 80tons preloaded earth anchor system.

Influence of wall stiffness on the surface settlement was investigated by applying the strut support for the various stiffness of walls. When the stiffness of R.C wall reduced to equivalent stiffness of the sheet pile wall, the maximum wall deflection doubled from 40mm to 78mm.

Relationship between maximum wall displacement and maximum ground settlement shown in Fig 12 indicate that the surface settlement patterns for various retaining wall systems are as follows.

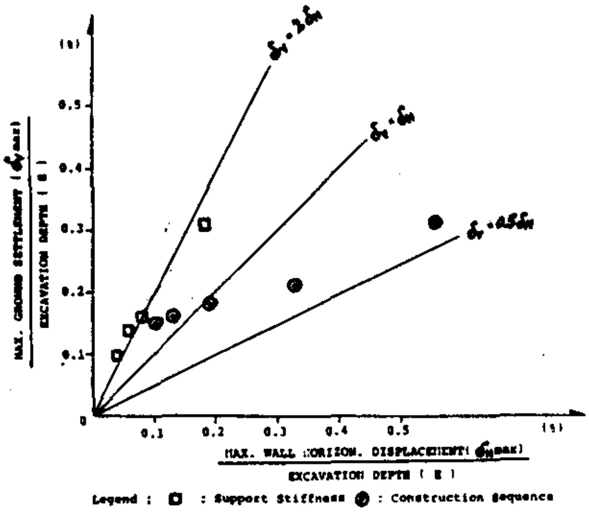


Fig 12

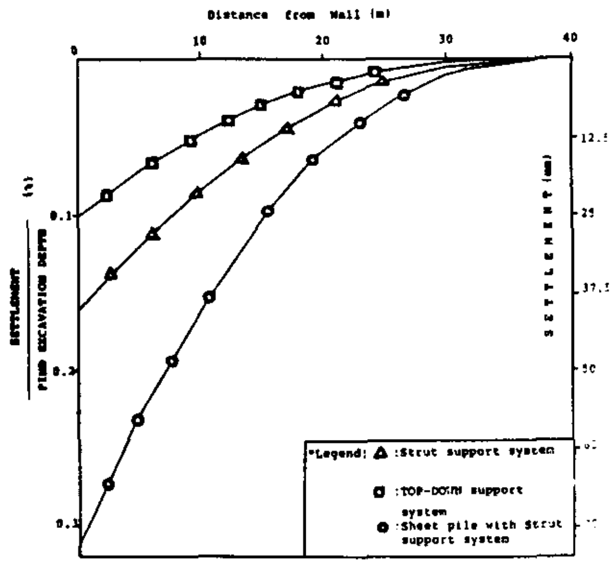


Fig 13

* Relationship between maximum wall displacement and maximum ground settlement

Rigid wall at early construction stage with ; $\delta_H > \delta_v > 0.5\ \delta_H$

At final stage of construction (Deep Excavation) ; $2\delta_H > \delta_V > \delta_H$

where, δ_H = maximum wall displacement
δ_V = maximum ground settlement

Fig 13 illustrates surface settlement behind diaphragm wall and sheet pile wall with various supporting system. The rigid diaphragm wall with basement slab supporting system in the Top - Down method can dramatically improve the surface settlement which should be well within design limits.

CONCLUSIONS

Based on the above results the following conclusions may be drawn.

1) The elasto-plastic analytical results shows good agreement with the field observed data. The elastic based model underestimates wall deflection greately.

2) The observed and analytical earth pressure distribution behind the rigid wall are quite close and in many case identical to the classical Rankine theory particularly near the final excavation level.

3) The displacement of diaphragm wall was significantly influenced by the stiffness of supporting systems.

The maximum wall movement of 19 mm, can be reduced to 7 mm by employing basement slab support of Top - Down method .

The wall designed with highly pre-loaded earth anchor support was found to be equally effective in controlling the wall movement.

4) The results of the influence of wall rigidity indicate that the rigid diaphragm, the maximum horizontal movement and surro[illegible] surface settlement were 50% less than that of the sheet pile wall.

5) The ratio of vertical settlement (δ_v) to horizontal wall displacement (δ_v) found in this project was as follow.

- Rigid wall for shallow excavation (early exc. stages)

$$\delta_H > \delta_V > 0.5\delta_H$$

- Rigid wall for deep excavation (final exc. stage)

$$2\delta_H > \delta_V > \delta_H$$

The total volume of deflected wall represents the amount of surface settlement, not the maximum wall displacement value.

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