In Situ Reinforced Earth—An Approach for Deep Excavation

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Introduction

Rational approach for projects involving deep open excavation requires the ability (*i*) to assess whether the proposed excavation is feasible under various imposed constraints (*ii*) to device new methodes of construction and (*iii*) to predict settlement, subsidence and other lateral movements. Peck (1969) has provided an excellent review of the several methods employed in geotechnical engineering practice. Of the several methods in vogue, strutted excavation with lateral bracing is the earliest and the most commonly employed method (Lambe 1970, Lambe et. al. 1970). Other methods of recent origin uses sheet piling and anchored bulk heads (Sowers & Sowers 1967) tied back supports (Mansur 1970), diaphragm walls with slurry trench techniques (Gerwick 1967, Kapp 1969). In the final analysis, the chosen method must take into consideration the structure-soil behaviour and the factors responsible for earth pressure so as to provide required strength and stability, appropriate flexibility or rigidity to the system. The method adopted for carrying out deep excavation for housing furnances in an existing Hanger of Hindustan Aeronautics Limited, Bangalore, has been discussed in the article.

Salient Features of the Project

Hindustan Aeronautics Limited, Bangalore Complex, proposed to commission Stein Atkinsen Furnace inside the existing Sheet Metal Hangar in Aircraft Division for purpose of Solutionising Aluminium Alloys. The furnace is an aircirculation furnace designed to work at a maximum *temperature of 550°C with an accuracy of* $\pm$ *1°C.* The existing Hangar is covered with A. C. Sheets over steel trusses spanning 30m, supported over *RCC* columns. From the standpoint of working requirements, the furnace weighing about 25 *MT* is required to be located at 2.5 m away from the load bearing columns which are founded about 1.5 m below the existing floor. The furnance requires a head room of 13*m* and hence a pit of 7.4m is required to locate the quenching devices. Taking into account the requirement of thickness of base concrete, the lowest level of excavation is about 8 metres below the floor level.

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Allowing for the thickness of retaining wall around the foundation, the prosposed excavation which is L-shaped, has a plan area of 140 square meters and occupies a position of the hanger with the rest of the area with already installed machinery under operation. Figure 1 shows the plan and cross-section of proposed excavation along with the disposition of adjoining structure.

Any solution for the problem for excavation for the foundation for such a depth will necessarily have to take into consideration (1) the effect of excavation on the existing structure as the location of the proposed foundation is in close proximity to the existing load bearing columns of the hangar (2) the effect of lowering the ground water level needed for carrying out the excavation, on the adjoining foundation, so as to arrive at an economically feasoble method of carrying out the excavation under the imposed constraints including the avoidance of noise and vibrations, elimination of lateral struts and to carry out unhindered work.

The conventional type of excavation of providing a gradual scope in effecting transition of levels was not tenable in the instant case due to inadequacy of lateral space (the other operations in hangar around the site are not to be disturbed). With the difficult ground water conditions (the natural water table being only 2.2m below floor level), open excavation may reselt in slippage of overburden, thus endangering the existing structure. Indequate space, proximity of existing columns and wall of the hangar, insufficient headroom within the hangar preluded the use of other known methods of excavation. The problem, therefore, needed scientific study of site conditions examination of alternative methods before

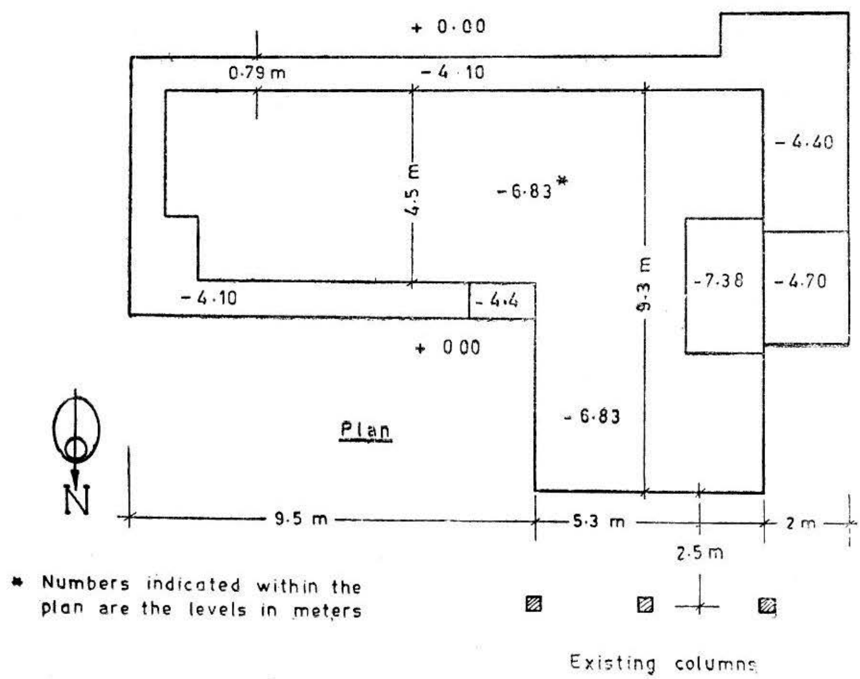


FIGURE 1 *a* Plan of excavation

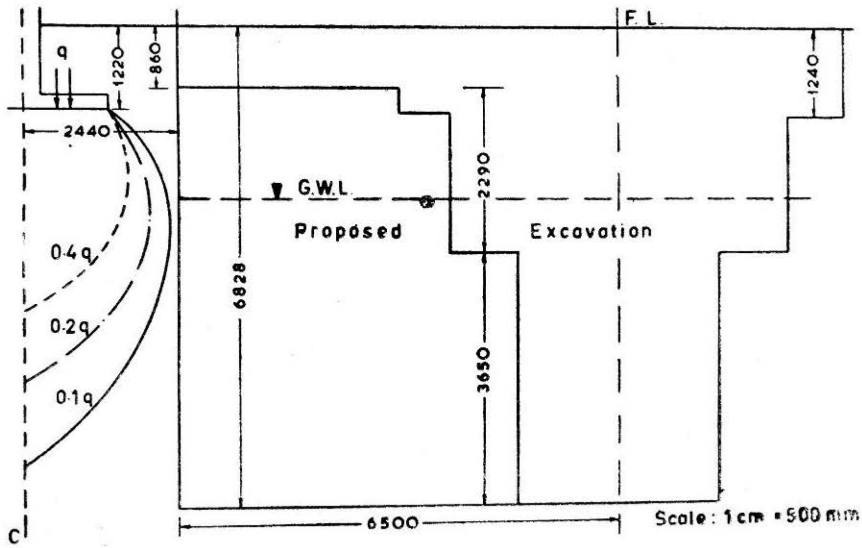


FIGURE 1 *b* Cross section close to boundary of R.C.C. column P_1 and stress distibution.

arriving at a solution that provides the required strength and stability, appropriate flexibility to the system and case in construction.

Site Conditions

Since no information was available about the substratum conditions, three locations were identified for trial pits within the area of the proposal excavation inside the sheet metal hangar. two trial pits were taken upto significant depths within which the lateral stresses due to column loading meritted examination.

Thin wall tube drive sampling was resorted to obtain undisturbed samples for strength and compressibility characteristics. It was noticed that below ground water table (2.2m below floor level) excavation could not be carried out easily and at times lateral slipping of soil were noticed. No doubt this was party due to not effectively lowering the ground water level ahead of the excavation. The soil is essentially sandy silt of medium to law plasticily. Locally this is termed as 'Sudde'.

Tables 1 and 2 show the in situ and index properties and strength and compressibility of soil.

Analysis of the Problem

As a result of the lowering of the ground water level during excavation & subsequently constructing the foundation structure, the safety of the column of adjoining structure need be examined with respect to the following two aspects:

(*i*) Settlement—As ground water table need necessarily be lowered by heavy pumping and maintained well below the bottom of the excavation for fairly a long period, the submerged density of soil within the signifi-

TABLE 1

In Situ and Index Properties

	Location 1	Location 2
In Situ Properties		
Depth of Sampling (m)	2.8	2.5
In Situ bulk densits (gm/cc)	2.00-1.90	2.00-1.90
Natural moisture content (%)	20-21	25-26
Dry density (gm/cc)	1.66-1.58	1.59-1.52
Index Properties		
Liquid limit (%)	49	64
Plastic limit (%)	20	24
Plasticity Index (%)	29	40
Particle Size Distribution		
Gravel (%)	1	4.6
Sana (%)	43.2	39.8
Silt (%)	46.6	43.6
Clay (%)	9.2	12.0

TABLE 2

Mechanical Properties

	Location 1	Location 2
Strength Characteristics		
Consolidated Undrained Test (*CIU*)		
c_{CU}	0	0
ϕ_{CU} (degrees)	28	28.5
Consolidation Characteristics		
Coefficint of volume compressibility		
(*i*) m_v (ft²/ton)		
(*a*) 0.25—0.5T/Sq. ft.	0.092	0.0670
(*b*) 0.5—1.0T/Sq. ft.	0.040	0.0600
(*ii*) C_c (Range 0.5—1T/Sq. ft.)	0.110	0.180
(Range 1—4T/Sq. ft.)	0.210	0.210

cant depth for the portion below the ground water level, will increase to the bulk density. This resulting changes in effective stresses were considered the computations of settlement of existing column loads. Considering the compressibility of the, soil, the corresponding maximum additional settlement was of the order of 1 cm (0.39 inches). Since this order of settlement, even if it were to be realised was regarded not to effect the structure, no special methods were resorted to prevent this settlement.

ii) In this problem the possibility of lateral flow of the soil during excavation and the associated stability posed relatively a serious problem. Apart from ensuring the stability of adjoining structure, excavation from 2 to 8 m. depth had to be carried out with the lowering of ground water level from 2 to 8 m. below floor level for sufficiently longer periods till the foundations and the retaining walls for furnaces are constructed.

Examination of Alternative Methods

In normal construction activity as excavation progressed the lateral movements are arrested by struts with or without pre-stressing. This could not be resorted to due to larger width and moreover lateral supports were not desired as they tended to obstruct subsequent construction activity. Other possible method of avoiding lateral supports for the entire width was to resert to tied back construction. It was not possible to resort to this due to inadequate space between the adjoining structure and boundary of excavation.

Instead of conventional sheet piling and lateral bracing anchored bulk-heads meritted examination for the proposed excavation. The sheet piling had to be driven to depth of 6m metres with the provision of suitable anchors. Driving sheet piles inside the hangar were not practicable due to the necessity of using heavy driving equipment with associated noise and vibrations, inadequate head room for smooth working of machinery. Further, provision of suitable anchors was not feasible due to restricted boundary conditions.

Adoptation of more recent method of costruction of diaphragm walls by slurry trench method was also examined. The typical diaphragm wall is from 0.6 m to 1.2 m in thickness with depths ranging from 6 m to 36 m. The trench can be progressively excavated in suitable streches from ground surface by power operated clamshell grab. Trench stability, till dia phragm wall construction, is ensured by bentonite slurry. This slurry ensures stability by its density and an efficient seal is formed. Subsequently reinforcements are set in and concreted so as to result in a wall strong enough to resist lateral deformation. The present state of-the-art is such that (Xanthakos 1979) this method has been extensively used to execute deep foundation excavations and underground excavation to provide rigid water tight wall that premits subsequent dewatering and excavation without causing settlements and ground water drawdown that might damage adjoining structures. It was not possible to resort to this technique primarily due to inadequate space for operation of plant and machinery. Moreover the diaphragm wall construction was not conforming to the subsequent foundation requirements for housing furnaces.

More recently to carry out deep excavations caissons are sunk to their final depths by internal excavation. The successful procedures include means for injecting bentonite as a lubricant around the periphery of the caissons. Due to irregular geometry of the proposed excavation and space constraints, this method could not be resorted to.

Any one of the above would have been reasonable alternatives if the proposed excavation were to be outside the hangar. Hence, a new innovative method was desired to solve the problem. The reinforced earth technique was examined.

In Situ Earth Reinforcement

The technology of reinforced earth, developed by Vidal (1969) has been well founded. The success achieved by construction of over 2000 projects is a testimony to the development of the associated technology. Considering, (*i*) the wide range of economics that have been achieved over alternatives (*ii*) that there are no practical limits on the magnitude and (*iii*) that structures can be constructed easily and quickly, reinforced earth technology is a major advance in the field of geotechnical engineering. (Nagaraj 1981). In Principle, the synergetic effects between soil and reinforcements arc computable by predictable friction and wholly relying upon the reinforcements to carry all the induced tensile stresses within the mass.

In situ earth reinforcement which has been developed and used in this investigation is based on the same principle as that of reinforced earth construction. This method mainly involves the introduction of reinforcements into soil system in their in situ state to convert a certain portion of soil mass into coherent gruvity mass which in turn resists lateral forces preventing lateral deformation. This mode of strengthening is similar to a natural phenomena of tall trees achieving their stability by converting a large portion of soil to act coheaently by spreading their strong roots in all directions. The possible method of mechanical reinforcements and stabilisation of plant roots have been studied in detail by Gray (1974). As far as the authors are aware of, this technique has not been employed for solution of a practical problem either within the country or outside except in one situation as reported by Lizzi (1977). The stregthening action is provided by basic element of soil reinforcement known as a Palo radice' (root piles) introduced along specific directions. The strengthened structure is known as 'Reticulated Pali Radice Structure'. The mode of in situ reinforcement, the functional requirements differ from the above approach.

Figure 1 b shows the cross section of the proposed excavation along with the location of the adjoining structure. The general principles of design are in the same lines detailed bo Lee (1973) to be followed in the case of reinforced earth retaining walls. The essential requirements to be satisfied are that the reinforcements need be strong enough to prevent failure by breaking in tension and frictional force adequate enough to prevent by pulling out. Tor steel rods of 25mm dia. were chosen in this case. The strengths of these rods are far higher than that needed. But the rods apart from satisfying the strength criteria had to essentially satisfy the requirements of drivability without any bucklining. The second requirement Viz., frictional component requirement can be adequately satisfied with proper design.

In Principle the lateral earth pressure developed at any depth was equated with suitable factor of safety to maximum tensile resulting force developed due to friction between the soil and surface area of reinforcing rods at that depth. In the computation of restoring forces two variables need be incorporated i.e., the horizontal and vartical spacing of rods and the co-efficient of friction between soil and reinforcement, on the assumption of the coefficient of friction between the earth and reinforcement to be 0.5 tan ϕ where ϕ is the undrained angle of shearing resistance of soil as per the prevailing practice (Schlosser at et al 1974). In addition, examination has been made against tie pull out. For this purpose the total frictional resistance at any depth is equated to corresponding lateral forces due to active earth pressure, that are likely to mobilize. According to the assumption made in the conventional reinforced earth construction the length of reinforcements beyond the Rankine zone are only to be considered. In this investigation this assumption has been violated as it was not practicable to drive very long lengths of reinforcements. Entire length of reinforcements was considered to be effective. The above analysis resulted in arriving at the following construction details:

(*i*) diameter of the tor or any deformed bar as 25 mm.

(*ii*) the minmum length as 225 cms.

(*iii*) the spacing in both horizontal and vertical layers as 22.5 cms.

Construction Procedures and Details

The sequence of construction was essentially in steps of advancing the excavation by about 0.5 m. and immediately strengthening the same by horizontal driving the reinforcing rods before the next stage of excavation is undertaken. Since the excavation had to be carried out below ground water table, inspite of continuous pumping it was necessary to prevent surface dislodging of the saturated soil mass. This was taken care by the use of wooden planks 2 m. long and 22.5 cms. wide with predilled holes at 22.5 cms. spacing along the line at the half of the width, as skin elements. As these planks were not intended to resist ony lateral stresses no elaborate arrangements was required to fasten this to the rods. Figure 2 shows the cross-section and front elevation details of the reinforcements and disposition of skin elements. Figure 3 to 5 show the methods of

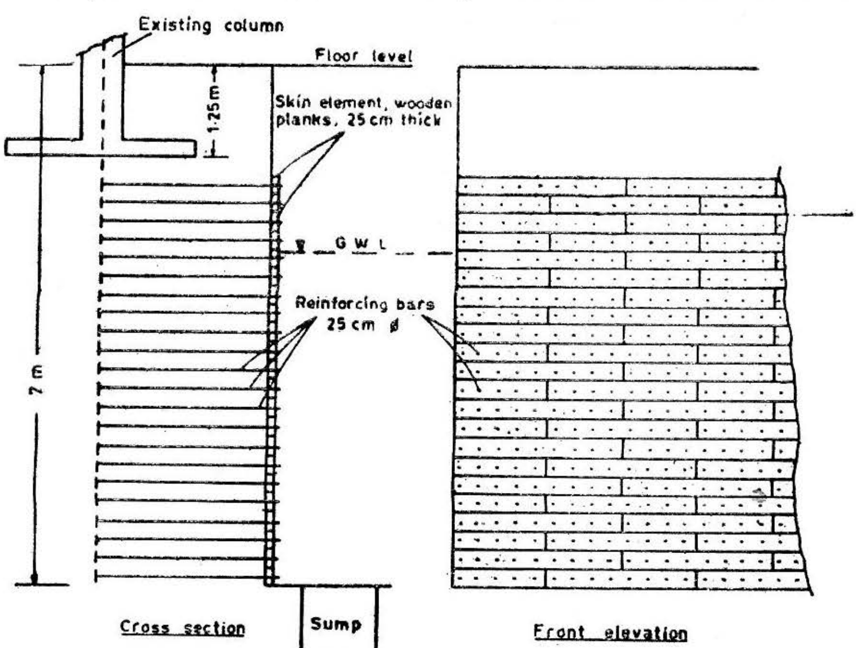
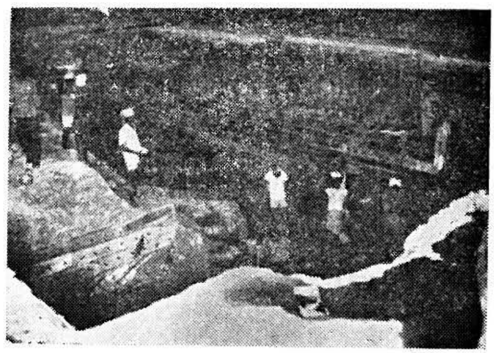


FIGURE 2 In situ reinforced earth details.

placing the skin elements and various stages of driving the reinforcement. Although it was possible to drive the rods of 225 cms. length horizontally mannually with hammer, other methods of easily driving at a faster pace was n ecessary to complete the project in time. Use of pneumatic gun working on a compressed air of 80 psi. was found to be quitehandy. It was possible to drive each rod to its final position within 4 to 5 minutes. By providing the sumps whose depths were always lower then 1m. from the corresponding depths of excavation and continuously pumping the inflow of water it was possible to handle the ground water problem satisfactorily. The pullout tests conducted on driven rods clearly indicated that the frictional resistance was very close to that of soil to soil. In the absence of earlier data on the pullout resistance of tor steel driven into soil, as per the current practices of reinforced earth construction 0.5 tan ϕ was assumed in the analysis of the problem and designs. This in a way turned out to be on the conservative side.



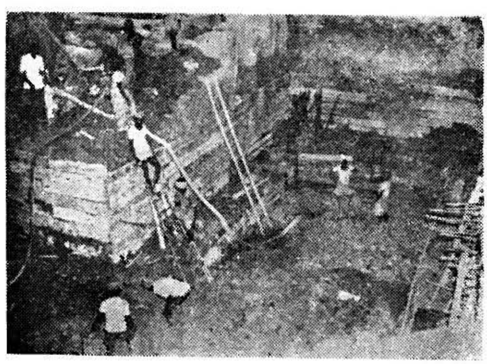


FIGURE 3 Earlier stages of progressive excavation.

Constant monitoring of levels of the floors, adjoining columns and lateral movements were carried out. No subsidence of the floor or lateral movements were noticed even after reaching the stipulated depths of excavation. To effect plane surface for providing water proof asphalt layers 3" (7.5cms.) burnt brick layer was constructed adjacent to the wooden planks and the projected rods. At regular intervals horizontal nominal reinforcing rods were tied to projections of in situ reinforcements to obtain a monolithic thin brick wall for subsequent water proofing. Having thus established a coherent ravity mass for the furnance posed no stability problem.

FIGURE 4 Different stages of horizontal advancing reinforcement through wooden planks as skin elements.

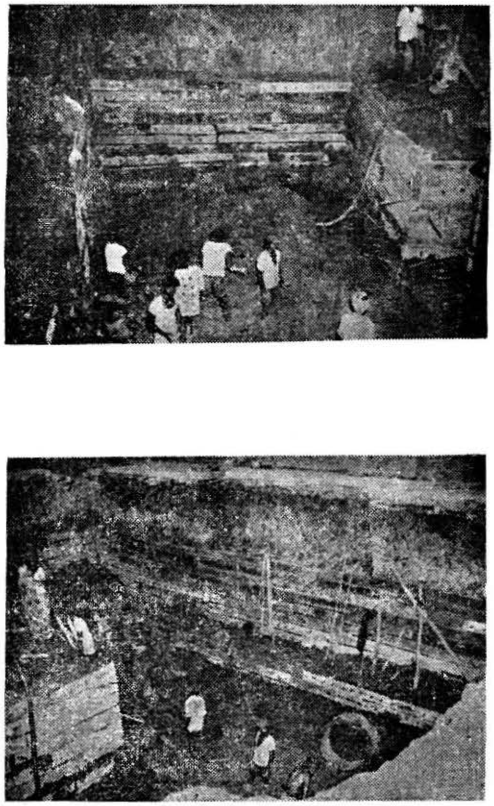


FIGURE 5 Advanced stages of excavation

Conclusions

By resorting in situ earth reinforcement deep excavation could be successfully carried out under adverse soil conditions, below ground water conditions simultaneously ensuring the safety of the adjoining structure. Properly disigned and external reinforced earth excavation with skin elements does not require any additional structure like retaining wall purely from the view point of retaining the earth.

This innovative proedure which has been tried in the field for the first time in the country, besides resulting an order of economy of 50 per cent over the alternatives has been completed in record time of 40 working days. Bseides this method satisfied other constructional restraints such as (*i*) avoidance of using heavy equipment inside the sheet metal hangar (*ii*) space costraints for employing other conventional methods (*iii*) avoidance of noise and vibrations for uninterrupted functioning of the workshop of sheet metal hangar.

In this invenstigation the lengh of reinforcements driven did not satisfy the principles laid for conventional reinforced earth construction i.e., the lengths beyond the Rankine zone were to be considered in stability analysis. This merits further investigation as to the need for adherence to this reqirement or examination of stability considering the whole volume of reinforced earth as coherent gravity mass. More data need be generated regarding the friction mobilised between deformed bars and soil.

This field of in situ reinforced earth is still open to the contribution of other researchers and designers such that the potential of this method in solving other problems such as enhancing the stability of natural slopes, execution of safe steep excavations gets exploited.

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References

GERWICK, B. C. (1967), 'Slurry Trench Techniques for Diaphragm walls in Deep Foundation', *Civil Engineering*, ASCE, Vol. 37, pp. 70-72.

GRAY, D. H. (1974). 'Reinforcement and Stabilization by Vegetation', *Journal of Geotechnical Division*, ASCE, Vol. 100, GT-6, pp. 695-698.

KAPP, M.S. (1969), 'Slurry Trench Construction for Basement Wall of World Trade Centre', *Civil Engineering*, ASCE.

LAMBE, T. W. (1970), 'Braced Excavation', Proc. Sp 1. *Conf. on Lateral Stresses in the Ground and Design of Earth Rotaining Structures.* ASCE.

LAMBE, T. W., WOLFSKILL, L. A. and WONG Ing. H. (1970), 'Measured Porformance of Braced Excavation', *Journal of Soil Mechanics and Foundation Engineering Division* Vol. 96, SN-3, PP. 817-835.

LIZZI, F. (1977), 'Practical Engineering in Structurally Complex Formations (The In Situ Reinforced Earth)', *Int. Sym. on the Geotechnics of Structurally Complex formations*, CAPRI, Vol. 1, pp. 327-332.

MANSUR, C. I. (1970), 'Tie Backs in Clay to Support Sheeted Excavation', *Journal of Soil Mechanics and Foundation Engineering Division, pp. 495-509,*

NAGRAJ, (1981), 'Reinforced Earth—Priciples and Applications', Under Review for publication in Journal of *Indian Geotechnical Journal.*

PECK, R. B., (1969), 'Deep Dxcavations and Tunnelling in Soft Ground', State of-the-art report, *Seventh International Conference on Soil Mechanics and Foundation Engineering*, Mexico, Vol. 3, pp. 225-290.

LEE, K. L., ADAMS, R. D. and VAGNERON, (1973), 'Reinforced Earth Retaining Walls', *J. of Soil Mechanics and Foundation Engineering Division*, ASCE, Vol. 99, SM-10, pp.745-764.

SDHLOSSER, F and ELIOS, V. (1978), 'Frinction in Reinforced Earth,' ASCE spring convention—Pittusburg—April 24-26,1978 pre print 3274.

SOWERS, G. B. and SOWERS, G. F. (1967). Failure of Bulkhead and Excavation Bracing, *Civil Engineering*, ASCE, Vol. 37, pp. 72-77.

VIDAL, H. (1969). The Principle of Reinforced Earth, Highway Resarch Record No. 282, pp. 1-16.

XANTHAKOS, P. P. (1979); Slurry Walls—MeGraw Hill pp. 622.