Simpler Techniques for Ground Improvements

by

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Introduction

Economic Considerations in choice of technique

Usually the choice of the construction technique is based on Comparison of costs at current prices. Similarly, while choosing a design concept the least cost alternative is preferred after making due allowance for meintenance cost, service life and durability. It is not realised that costs at current prices do not always reflect supply constraints with regard to materials, equipment and skilled labour due to imperfections of the market machanism and Government controls. Techniques used by economists for projection of future costs always involve a subjective element. It is, therefore, difficult to justify research projectc on the basts of projections of future savings when the cost comparisons at current prices do not reflect a significant saving on the other hand. It takes several years between the completion of a research study and its applications. In the meanwhile relative prices may change radically. As a consequence, research may be directed into unproductive channels and may fail to yield desired economic benefits or serve to overcome supply constraints.

The energy crisis has resulted in a sudden and rapid increase of construction costs especially the cost of transportation, fuel and electrical energy; and cost comparisons made only a few years ago have lost their relevance.

It is now quite evident that we have to find a way of living with energy shortages and there is no prospect of a radical improvement in the availability of energy per capita in the next 20 years at least in the developing countries. We are on the other hand using or adapting techniques which have been developed in countries which have better endowment of energy resources of which command a relatively higher share of the World Energy Resources.

When serious energy shortages were first experienced a few years ago, I started looking at the energy consumption in construction industry. I found that it is necessary to consider the energy requirements for manufacture of construction materials and equipments, apart from the direct consumption of energy in the construction process. I soon realised that a new technique or a design concept capable of bringing about major saving in energy would tend to become progressively more economical

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with passage of time. I, therefore, started searching for techniques which have the potential for a drastic reduction in energy consumption and at the same time offer a significant saving in the cost at current prices.

I would like to add a note of caution regarding cost comparisons based on analysis of quoted rates for recent tenders. One should not rely on such cost comparison without making an evaluation of the margin over basic costs and testing the sensitivity of comparative costs to variations in supply conditions. In the first instance, basic cost of the main factors should be estimated such as material, equipment, fuel or energy, transportation and labour according to standard procedures and norms. Material and labour cost would be based on current prices and norms of consumption of material and average labour productivity. Fuel or energy cost can also be similarly estimated by examining the consumption for various categories of equipment with due allowance for the intensity of utilisation of the prime mover e.g., the energy consumption per horse power/hour of heavy earth moving equipment cannot be the same as light handling equipment or mixing plant. Equipment cost should be based on norms of physical depreciation based on working life and current rates of interest, insurances and taxes. It is desirable to examine the margin between aggregate basic cost and quoted price. Usually under normal conditions, these tend to approach normal expections of profits and overhead costs. When the margins between the aggregate basic cost for various factors and the quoted price appear to be abnormal, the cost structure and the cost trends should be examined to find out whether the current quoted prices are influenced by short term scarcity of skills or peculiar market conditions. After considering the margins over the basic costs, costs comparison should be further examined with regard to their sensitivity to variations in the basic costs due to market fluctuations or shortages.

Estimation of Energy Consumption

Energy consumption should be considered not only in terms of the fuel consumption or consumption of electrical energy in the construction process but also in terms of energy consumption for materials required for construction and in processing of materials, transport, haulage, handling and placement as well as energy consumption in repair and manitenance of construction equipment and energy consumption corresponding to physical depreciation.

Norms for estimation of energy consumption should be worked out in the first instance in terms of energy consumed as fuel (diesel or coal) and electrical energy per unit for daily production of the construction team or unit of equipment. Estimate of energy consumption by construction equipment should include equipment for processing, handling and placement and these should be further grouped into heavy, medium and light equipment. Horse power per worker can be used as an index of categorisation of equipment as heavy, light or medium. It is also necessary to consider the intensity of energy use while working out the norms for energy consumption of equipment.

Energy consumption in materials of construction can be a major component of cost if energy intensive materials such as cement, steel or polymers are used. Materials from renewable sources such as natural fibres, bamboo, timber have evidently great potential for saving energy. Energy consumption in processing and haulage of aggregates can be reduced by using local low grade aggregates and soil with a minimum of processing. Use of lime and fly ash can also contribute to significant energy saving when they replace cement.

Energy consumption and mixing in compaction is often not considered while evaluating alternative construction techniques. For example, soil stabilisation may require mechanised plant for mixing and compactions. Alternative technique which tolerates moderate variations in mix proportions and if mixing in wet condition can be permitted would also result in large saving in energy consumption.

Energy consumption data for construction processes should thus be compiled on the basis of following parameters:

—Horse power hours of equipment per unit of production. This may be further sub-divided into heavy and light equipment.

—Ton-kilometer of haulage for various materials by different modes of transport such as railway, truck, or manual labour or animal power.

The energy consumption in terms of diesel or any other fuel can be worked out for each category of equipment. It is possible to make an evaluation of the physical depreciation of plant and consumption of materials for repair. By knowing the nature of the manufacturing process for the machinery, it should be possible to obtain an energy equivalent for an equipment of a rated Horse power capacity for each of the equipment categories.

The energy consumption for various modes of transport as well as the equivalent energy consumption corresponding to the capital cost of transportation should be considered. Thus the transportation components can also be converted into energy equivalent.

With regard to construction materials, this can be divided into following broad categories:

—Steel and metals.

—Cement, lime and inorganic cementacious materials.

—Polymers and petrochemicals.

—Natural fibres, timber and material from renewable biomass.

After considering the consumption of material in each category, it should be possible to assign energy equivalent to each of the category of the above materials, in terms of a common material in each category such as steel, cement, HDPE. The energy saving potential of materials from biomass sources is obvious. However, the supply of biomass material could be a constraint and a major improvement in the conventional techniques of biomss materials is necessary to improve the utilisation efficiency.

Problem of Transition

1.3.1 Research and Development effort would be productive, when a preliminary analysis indicates significant savings in costs at current prices and there is good potential for major savings in energy. However, it is necessary to consider the problems of transition. During the initial stages of introduction of a new technique, costs would be high when applications are made on a small scale. Realistic projections should, therefore, be made of trends of costs by taking the economics of scale into consideration. Development effort is also another major element of cost. Saving must be sufficient to justify the developmental effort and sensitivity analysis should be carried out to verify whether the estimated savings can be sustained over a sufficiently long future period. Introduction of new techniques is facilitated if a step by step transition is technically feasible so that some of the currently available materials and equipment are used while others are progressively substituted by the new materials and equipment. As far as possible, the transitional techniques should not require special skills which can be acquired only after a long period of training.

Brief resume' of previous work

In my early work my attention was concentrated on ground improvement; the relevant research work was carried out mainly from 1970 to 1974 while field applications were made from 1970 to 1978. Performance data was, therefore, already available by 1976.

Around 1974, I realised the crucial role of energy as the principal cost factor. My subsequent research was, therefore, concentrated on materials and construction methods with emphasis on the energy consumption. The research findings were published in several National and International conferences. The papers could be grouped as 2 papers on bamboo, 2 on soil improvement, 2 on soil reinforcement and 1 paper on appropriate construction technology and 1 paper on reinforced stabilised soil. The detailed list of reference is annexed.

General Scheme and arrangement of the lecture

The presentation is arranged according to the three main topics i.e., ground improvements, soil reinforcement, and soil stabilisation. Under each topic, the various available or known techniques are described and a brief resume is presented of design methods which would help to select the least cost solution. This is followed by a comparative evaluation of various techniques with regard to their performance cost and energy consumption. In the concluding paragraphs of each section, suggestions are made regarding the choice and development of energy saving low cost solutions and the research needed to introduce the proposed techniques or to extend the use of known techniques which have the best potential for saving energy.

Methods of Ground Improvement

In this chapter, the three most common ground improvement techniques will be considered comprising :

—Consolidation by preloading and vertical drains.

—Strengthening of the soil by granular columns and other replacement methods such as lime columns.

—Compaction of deep soil layer insitu by heavy tamping, vibroflotation and other vibratory equipment.

The main purpose of this lecture is to review the various techniques and examine them with regard to their suitability for Indian conditions and to identify the areas for developmental effort and research. Evaluation criteria will be set up and the various techniques will be compared with regard to cost and energy consumption. In the field of ground improvement, I cannot overemphasise the importance of a well planned investigation programme which should be followed by initial trials during construction and monitoring of post construction performance. (Datye and Nagaraju 1975). It is regretted that limitations of the space and time do not permit a detailed presentation of the theoretical aspects of design along with observational data from typical case histories. Therefore, only a brief resume of the theories will be presented.

It follows from the above discussion that the selection of the most suitable technique on the basis of the cost reduction and energy saving should proceed through the following steps :

1. Identify and review alternative techniques.
2. Compile energy consumption data and establish values of parameters for energy consumption of various techniques.
3. Define performance goals and establish evaluation criteria.
4. Conduct design studies to determine the dimensions and arrangements of altrnea tive systems based on various techniques.
5. Compare costs of alternatives at current prices.
6. Compare energy consumption for various alternatives.
7. Evaluate the alternatives with reference to long term (Performance and Cost) trends.

Various ground improvement techniques will be considered generally according to the above sequence. After making a general review of known techniques, alternative will be examined with regard to specific applications. In the concluding sections, research needs will be identified; the prime consideration in the choice of research themes being the potential for energy saving and corresponding long term economic benefits.

Energy consumption in ground improvement

Components of energy consumption are described below, with reference to the various techniques mentioned in section 2.1 and the procedure for making an indirect and approximate evaluation of energy component of machinery depreciation and repair, as well as material and transport costs.

Energy consumption can best be estimated in terms of the horse power of a typical installation. Corresponding data on labour employed for operation of one unit of equipment should also be collected. These basic data can then be interpreted to arrive at the hp/hrs. required for installation of one linear meter of the device used for ground improvement or treatment of one sq. m. of the ground surface up to a stipulated depth.

These data can then be used for estimating the energy consumption for operation of equipment in terms of diesel or electrical energy. The fuel consumption of equipment depends on the size and intensity of its use and a due consideration of this factor is necessary. A further allowance should be made for the energy consumption corresponding to depreciation and replacements of construction plant. Approximate values of this component are used in this study. While estimating the replacement and depreciation component, the equipment should be categorised as light, medium and heavy. The categorisation would be governed mainly by H.P./Worker and the nature of use e.g., track mounted equipment has usually a higher rate of depreciation and repair as compared to semi-stationary winches or cranes moving on rails.

The energy consumption in manufacture *should then be considered* e.g., this may be quite appreciable for the fabric drains. Since the processes of manufacture of polymers are complicated, it is convenient to estimate energy consumption of materials in terms of kg equivalent of HDPE or PVC consumed per meter of the drain. For any specific polymer, conversion factors to arrive at kg equivalent of HDPE may as a first approximation be taken to be proportionate to the current cost exclusive of excise duty.

A component of energy consumption is often neglected in the energy consumed in production, processing and haulage of sand, stone and crushed aggregate. Approximate values in this study are considered to be adequate for the present purpose.

Assumptions adopted for cost analysis are stated below :

—Depreciation 20 per cent of capital cost per year.

—Fixed capital charges : 50 per cent of capital cost per year comprising depreciation 20 per cent, Repairs 15 per cent, Tax, insurance and interest 15 per cent.

—Working hours for machinery 2000 per year.

—Energy equivalent of depreciation is computed by dividing 50 percent of the depreciation by current price of diesel Rs. 3/- per litre.

—Energy consumption for transportation is estimated at 1 lit. diesel for 20 T-Km.

—Diesel consumption : 0.16 litre/HP-hour for heavy plant and reduced values are used for light plant according to experience.

—The energy equivalent of polymer is computed by dividing 50 per cent of the material cost by current price of diesel.

The above assumptions indicate an order of magnitude of the costs and are intended to provide a basis of evaluation of alternate technologies. Where the choice is sensitive to the energy consumption for any specific component, more detailed study will be necessary.

Ground Improvement by preloading and vertical drains

2.3.1 The strengthening and preconsolidation of weak and compressible soil by preloading is one of the most widely used methods for soil improvement. This technique is well suited for soil such as soft clay which undergoes large volume reduction and strength increase under sustained static load provided sufficient time is available for consolidation. By providing vertical drains, the time of consolidation is shortened. This technique is best suited to improve foundation soil subjected to area load such as embankment, tanks, and storage facilities. The emphasis here is on evaluation of recent developments and consideration that govern the choice of the technique of installation of drains and the analysis carried out for arriving at the least cost solution. With regard to theoretical basis of design and performance experience, reference should be made to recent comprehensive treatment of drains notably those of Johnson (1970a, 1970b, Bjerrum (1972), U.S. Navy (1971), Pilot (1977), Schlosser and Juran (1979), Akagi (1977, 1979), Hansbo (1979).

Recent developments

The following resume' of recent developments is based on the general report of Mitchel (1981).

Until a few years ago vertical drains of sand, typically 200 to 500 mm in diameter and spaced anywhere from 1.5 to 6.0 m on centres, were widely used. Installation was accomplished using a variety of techniques of both the displacement and non-displacement type. Displacement drains, while generally are less expensive and faster to instal, can disturb the surrounding soil. The resulting smear zone can impede drainage, and the disturbed soil may be weakended. These effects may not be as detrimental as believed earlier. However, owing to the possibilities of reconsolidation to a higher strength than the original, and the opening of cracks and fissures that fill with sand during installation and thereby increase the effective drainage area, Alkagi (1977, 1979) concluded that reliable data are lacking to establish whether non-displacement drains are indeed more effective than the displacement type.

A variety of prefabricated drains are coming into wide use abroad. These are not yet available in India. Band-shaped drains of the order of 100 mm wide by 1 to 7 mm thick are produced by several manufacturers. These drains can be rapidly installed to depths up to 50 meters by machines with special mandrels. Drain spacings of the order of one meter are typical. Both dynamic and static methods of installation are used. Prefabricated sand drains or sand wicks composed of sand placed within cylindrical fabric containers are also used.

Methods of Installation

Equipment for vertical drains can be categorised according to manner in which the tool for installation is advanced into the ground i.e., by jetting, by driving, with a hammer or by vibration or combinations thereof. Another basis of categorisation is the extent of soil displaced by the tool. Under Indian conditions, displacement methods are common for driving of large diameter i.e. 400 mm drains. The contractor can employ equipment commonly used for driven cast in place concrete piles. The size of the market does not justify deployment of special equipment; and, therefore, for larger jobs driven displacement type sand drains have mostly been used. Indian experience which is limited to moderately sensitive soils indicates that performance of displacement technique is satisfactory (Datye and Nagaraju 1975, 1976) and experience abroad seems to substantiate this view (Broms, 1979, Akagi 1977, 1979).

Precautions such as filling the casings with water before pouring sand, feeding water under pressure as the tube is withdrawn have been effective in ensuring continuity of placement of 400 mm drains.

Although vibratory driving equipment are not yet common in India, there is a good potential for introducing vibratory equipment. Apart from bringing down cost, vibratory equipment will improve the quality of installation since flow of sand will be facilitated and the hazard of necking will be eliminated.

Sandwick is an alternative method which has been used successfully for several installations in India. This system consists of a fabric tube somewhat like an elongated sock, in which sand is filled. The development work for this technique was carried out in India by Cemindia and Dastidar and for details, a reference should be made to previous publications of Dastidar (Dastidar 1969).

Drains of synthetic fabrics have note been introduced in India mainly due to the non-availability of material. However, an alternative viz., the rope drain which is a CBRI development has been used on several jobs. This is an energy saving method since small size mandrels can be used and the rope material consists of natural fibres such as coir. Unfortunately, performance data with relevant information regarding drainage capacity of rope drains are not available.

After a study of the cost factors in sand drain installations, I initiated in 1978 a modification of the installation procedure wherein the diameter of the sand drains was reduced to 200 mm while the hazard of necking was avoided by the use of a core of a small plastic pipe protected by bamboo strips that serve to restrain the sand material from moving upwards during extraction of the tube; continuity is assured and the inner plastic tube serves as a supplementary path of flow. An auxiliary shoe at the bottom prevents the bamboo strips from being pulled out and the strips in turn restrain the upward movement of the sand by shear. Necking is thus prevented.

10,000 drains have been successfully installed by this method. The procedure is illustrated in Figure 1. The range of applicability of this system can be extended by jetting methods, to facilitate driving of the tube and by developing a joint which admits of quick coupling and un-coupling of the casing tube in the field while the tube is being driven.

Evaluation Criteria

Alternative installation techniques for vertical drains should be evaluated in terms of total cost of ground improvement and/or energy consumption to achieve the desired performance. The performance goal for a vertical drain system is to achieve a stipulated degree of consolidation in a given period. In the evaluation of alternative drains system, estima tion of the time or consolidation is of vital importance. In practice it is not easy to make a sufficiently accurate estimate due to layering and anisotropy of the soil, the uncertainty of evaluating the smear effect and the influence of non-linear permeability and consolidation behaviour in respect of the drainage phenomena in the radial direction towards the periphery or external surface of the drain. The drainage capacity or well resistance is an important consideration which is sometimes ignored. (Annexure 3.

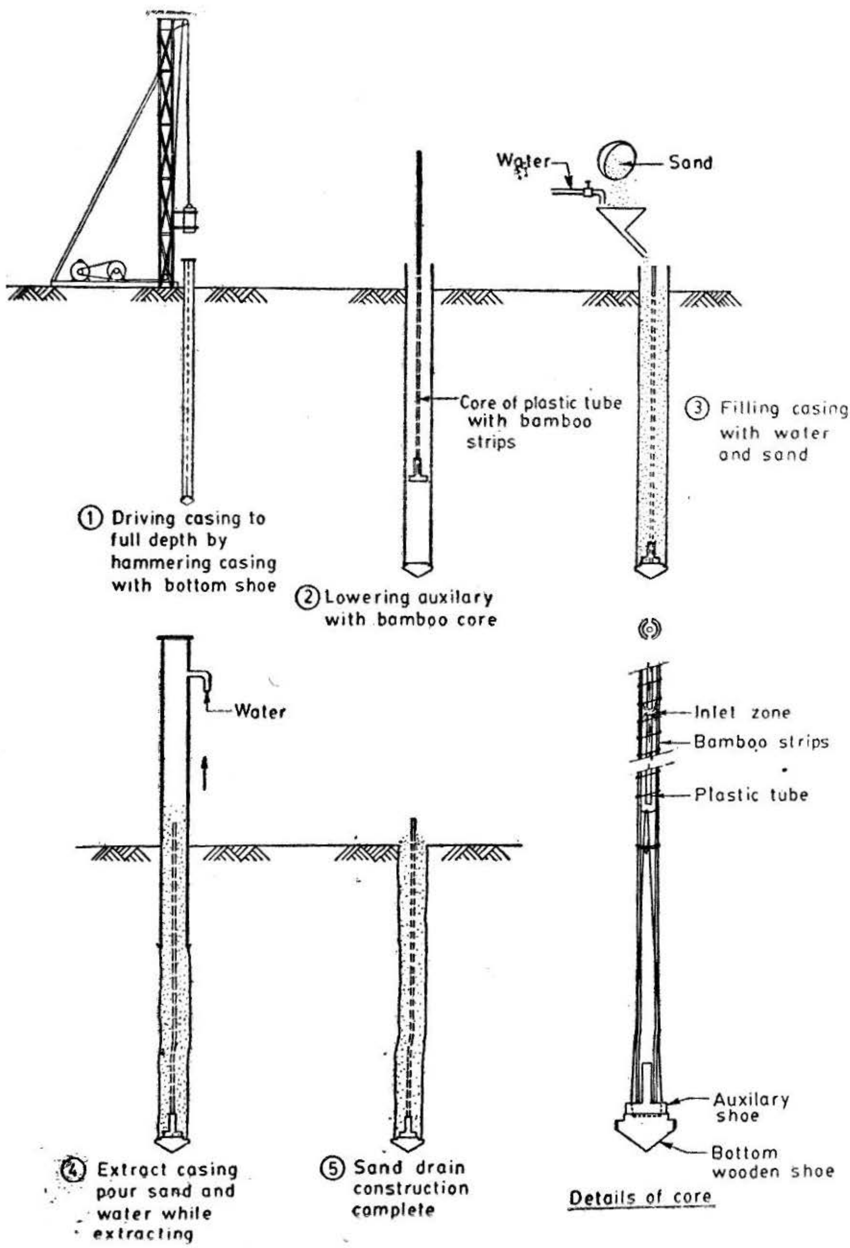


FIGURE 1 Sequence of installation of sand drains

Investigation programmes must be carefully planned and suitable procedures should be selected to ensure that a clear indication is available of the pattern of deposition of the sediment and the fabric of the insitu soil (Rowe 1972). The first step in investigation would be to categorise the soil with regard to thickness and extent of layers, and its fabric. This shall be followed by field tests such as permeability, lag time of piezometers and test plots, to establish representative values of C_H and C_V.

Regarding the smear effect, we have used the method of analysis suggested by Richart (1957) and Navy Manual (DM-7) and our limited experience substantiates the validity of this method. I would like to emphasise the importance and the need of an observation based approach for a reliable estimation of the insitu consolidation characteristics and the time of consolidation. Case history of the application of this approach was presented in a paper jointly contributed by Datye and Nagaraju 1975.

It must also be remembered that the cost of the ground improvement system consists of the cost of pre-loading and the cost of vertical drain installation. Hence increased cost of drains does not necessarily add considerably to the total cost of the ground improvement system. In some cases, the relatively higher cost of closely spaced drains may be offset by the comparative saving in the preload cost, since the reduced period of consolidation would facilitate repetitive use of fill material.

Comparative Evaluation of various sand drains techniques with ragard to cost and equivalent energy consumption

Results of the comparative study of various techniques are presented in Tables 1, 2. The techniques compared consist of sand drains of 400 mm diameter, 200 mm diameter, sandwicks of 65 mm diameter, and fabric drains with synthetic fibres. The comparative evaluation is made for two sites for a drainage system designed to achieve 90 per cent consolidation in 90 days. At site *A*, the coefficient of horizontal consolidation was 10

TABLE 1

Unit Cost of Sand drains, Sand Wicks and Fabric Drains

Cost Component		Drain diameter/width in mm				
		Sand drain 400	Sand drain 200	Sand Wicks 65	Fabric drain 150	Fabric drain 300
H.P.		250	35	15	250	250
Capital Cost	Rs.	15,00,000	2,00,000	1,00,000	15,00,000	15,00,000
Capital charge per hour	Rs.	375	50	25	375	375
Output/Hr. Mt.		10	12.5	20.0	120	120
Capital charge Mt.		37.5	4	1.25	3.12	3.12
Fuel+Lubricant	Rs./M	10	3	1.50	0.84	0.84
Labour Cost	Rs./M	5	3	2	4.17	4.17
Material & Royalty	Rs./M	10	3	4.25	6.0	12.00
Aggregate basic cost	Mt.	62.5	15	9	14.2	20.1
Quoted Rate	Rs./M	80	20	12	18.0*	25.2*

*Estimated.

TABLE 2

Comparison of Cost and Energy Consumption for Alternative Sand Drain Techniques
Depth of Sand Drain 10 Meters

Period of consolidation 90 days
Degree of consolidation 90%

Basic of Cost		Drain diameter width in mm			
		400 φ	200 φ	65 φ	150 mm wide fabric
SITE A: Ch = 1 m^2/year					
## Spacing of drains considering smear effect (2 R)	Mt.	1.60	1.40	1.10	0.7
Drains for 100 m^2	No.	45	59	96	238
Unit Rate	Rs./Mt.	80	20	12	18.0
Cost of treating 100 m^2	Rs.	36,000	11,800	11,520	42,840
Equivalent energy consumption	Ltrs.	2,606	592	572	3,231
SITE B: Ch = 10 m^2/year					
## Spacing of drains (2 R) with smear effect considered	Mt.	2.4	1.6	0.68*	0.68*
Drains for 100 m^2 area	Nos.	20	45	250	250
Unit Rate	Rs./M	80	20	12	18
Cost of treating 100 m^2	Rs.	16.000	9,050	30,000	45.000
Equivalent energy consumption	Ltr./diesel	1,164	803	1,224	3,623

* *Spacing limited by drainage capacity.*

Refer Annexure 3.

meters sq. per year while at site *B*, it was 1 meter sq. per year. Due allowance was made for the smear effect and the limitations of drain capacity were considered. The cost of sand drains of 400 and 200 mm diameter are based on quoted rates and the sandwick prices are also based on quoted rates. All rates are adjusted to a base price for 1980. Cost data from quoted prices are not available for fabric drains since this technique has not yet been introduced in India. The unit prices for fabric drains are, therefore, based on outputs reported abroad and the potential Indian outputs are taken at about 70 per cent of the performance abroad. Polymer cost for fabrics is estimated as Rs. 60/kg considering that the principal material will be polyester fabric.

The basis of cost comparison is furnished in Annexures 2 and 3. It may be noted that the cost considered in the study are relative costs and the main purpose is to provide a basis for a comparative evaluation. The prices are not to be considered as budgetary prices for economic evaluation of ground improvement system relative to other alternatives.

The conclusions that emerge from the study are very interesting. It is found that the 200 mm sand drains is economical for a wide range of conditions while the cost of sandwick and 200 mm sand drains may be very close when soil with low horizontal coefficient of consolidation are encountered. In layered soil with high values of horizontal coefficient of consolidation, the drainage capacity of sandwick may become a constraint. From the energy point of view, the 200 mm sand drain and san wick come very close for homogeneous soils while for layered soils with high coefficient of horizontal consolidation, the energy consumption for the sandwick is higher than the 200 mm sand drains.

The sandwick has the advantage of high outputs and case of field control. All the same, there is a good case for further developments and improvement of equipments for 200 mm sand drains especially for small jobs with depths under 15 meters.

With regard to fabric drains, the drainage capacity is a major constraint and unless very high outputs are needed, there is no cost advantage under Indian conditions. This is mainly due to high cost of polymer fabric relative to equipment and labour cost. The energy consumption for the fabric drain is also higher. This shows that there is hardly any case for introduction of fabric drains in preference to available techniques such as 200 mm diameter sand drains and sandwicks.

In Table 3, a comparative evaluation is made of the total cost of a sand drain and preload system. In the first instance, two alternative values of degree of consolidation were considered ; the main object being to evaluate the cost of preload for a system with a wider spacing of sand drains where the preload height had to be increased to achieve the same effective preload intensity even though the degree of consolidation was lower. Secondly, two alternative periods of consolidation were considered with a view to evaluate the benefit of a short period of consolidation which would permit repetitive use of the preload. The comparative study is valid only for the specific case considered; yet, it would be seen that there is no significant gain by reducing the period of consolidation unless the project schedule demands a shorter period.

Replacement by compacted granular material-stone columns

Techniques and Installation Procedures

In this technique, soft cohesive strata are replaced by granular material which is compacted by ramming or vibration. A vibratory poker is advanced by jetting and the granular backfill is added through the annulus formed around the poker by the water jetting. The

TABLE 3

Optimisation to ground improvement system considering total cost of sand drain and preload

Cost of consolidation for 100^3 plot			
Period of consolidation	Cost Component	Sand Drain	
		200 mm ϕ	200 mm ϕ
90 days	Sand drain cost Rs.	9050	4600
	Fill Height	6.5 m	8.5 m
	Total preload cost Rs.	16975	21825
	Sub-Total Rs.	26025	26425
45 days	50% preload quantity supplied at 25 Rs./m^3	8500	10913
	50% preload quantity re-handling at rate 10 Rs./m^3	3400	4365
	Total cost of preload	11900	15278
	Cost of sand drain	18100	9200
	Sub-Total Rs.	30000	24478
	Degree of consolidation	90%	70%

granular fill is compacted by vibration so that a compacted sand and gravel column is left behind as the vibratory poker is withdrawn. In the vibro composer method, a casing pipe is driven to the desired depth by a vibrator at the top. A sand and gravel charge is then introduced into the pipe, the pipe is withdrawn partly while compressed air is blown inside the casing to hold the sand and gravel in place. The pipe is vibrated down to compact the sand pile.

In the rammed stone column technique, the granular fill is introduced into a prebored hole and compacted by operating a heavy rammer through the bore hole. The cased bored hole may be advanced by conventional boring methods (e.g., bailor boring). Alternatively, a tube with a dispensable shoe can be driven to the required depth by using pile driving equipment. It should be noted that all the stone column techniques i.e., the vibro compaction, vibro composer, and rammed columns are self adjusting to the soil condition to the extent that enlargement of the column during ramming or vibration depends on the soil consistency.

Development of the rammed column technique and field control of compaction

In the initial stages, it was considered that a cased bore hole is essential to ensure that a clean column is formed which is free of contamination from the soft clay. Further compaction under wet conditions was preferred and the bore hole was filled with water to prevent sand blow. This resulted in dissipation of energy of the rammer and consequently the entire operation was slowed down and the total period of installation of the stone column including boring was as much as 12 and sometimes 15 hours.

Subsequently about two years ago, stone columns were installed through a driven tube with a dispensable shoe, In this method, the operations were simplified by operating the rammer at an elevation about 3 to 4 meters above the bottom of the hole. Due to the longer depth of filling in each operation, the installation was expedited and it was found that there was very little ingress of water into the cased hole with dispensable shoe. The diameter of the stone column was reduced to 400 mm; still the heavy pile/drilling rig was required for driving 10 meter long casings.

Trials have been carried out of installation of stone column through uncased bore holes where the contamination of the stone by the clay slurry is avoided by use of an enclosure of bamboo strips. In this system, the casing is installed for a short length which is so chosen that the compaction effect for a rammer operating at the bottom of the casing would extend upto the lower tip of the stone column. This system is found to be very rapid aad economical when the stone column length is limited to 8 to 10 meters. The various methods of installation of rammed stone columns are illustrated in Figure 2a and 2b.

Compaction Control

The degree of compaction that can be achieved depends, on several factors including the size of the hole, size, gradation and shape of the granular fill, depth of filling the weight and height of fall of hammer, number of blows and whether a dry or wet process is used. In general, the degree of compaction can be measured by a 'set' criterion i.e., penetration of rammer into the filled material for a given number of blows. Control on 'set' along with measurement of consumption of stone will ensure a uniform quality. A graded mixture comprising crushed stone of 23-75 mm size and medium sand (below 2 mm) will ensure freedom from segregation and better inter granular contact as the sand eventually work into the voids of the stone in a gap graded system. Initially it was thought that the campaction should be carried out in short 'passes' but recent experience indicates that the interval between the positions at which the rammer is operated can be increased. The compaction effect of heavy rammers is found to extend to depth of 4 or even 6 meters below the level at which the rammer is operated. Drop hammers are found to be most effective when the water level in the bore hole is just about the top of the granular fill. Diesel hammers are not found to be useful since they cannot develop the reaction required for efficient operation.

Design Approach

The stone column is essentially a system of soil reinforcement with the additional advantage of providing a drainage path. The stone column has the ability to adjust itself to the applied loads and to redistribute the load where stress concentrations occur. This is because there is no collapse

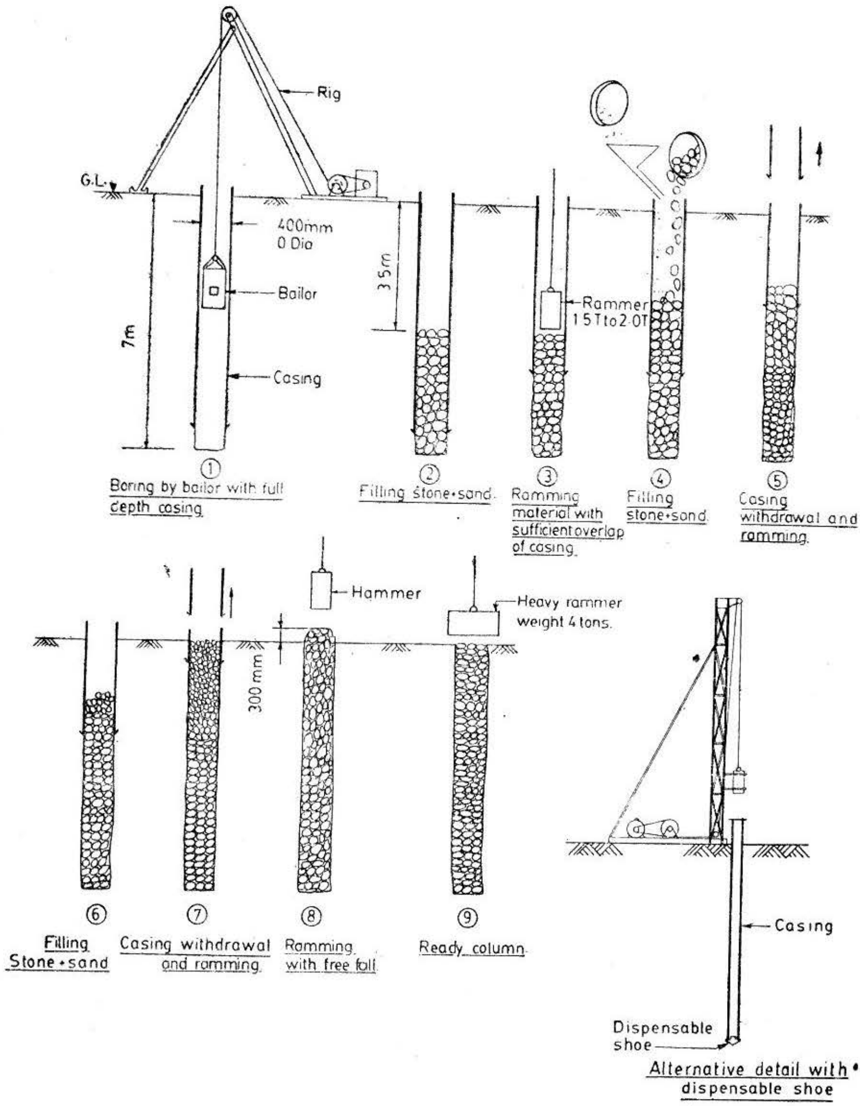


FIGURE 2*a* Installation methods rammed stone columns through cased bore holes

as such but only increase of deformation associated with bulging when the critical vertical stress level is exceeded. If a granular pad is placed over the stone column, then the process of redistribution is facilitated.

Design of stone column system involves two aspects, viz.

—Estimation of the yield load or the yield vertical stress at various elevation and choice of factor of safety.

—Settlement analysis.

Theoretical analysis of stone columns with regard to the yield load and settlement is complicated due to the inter-action of several factors. It appears prima facie feasible to use cavity expansion theories for estimation

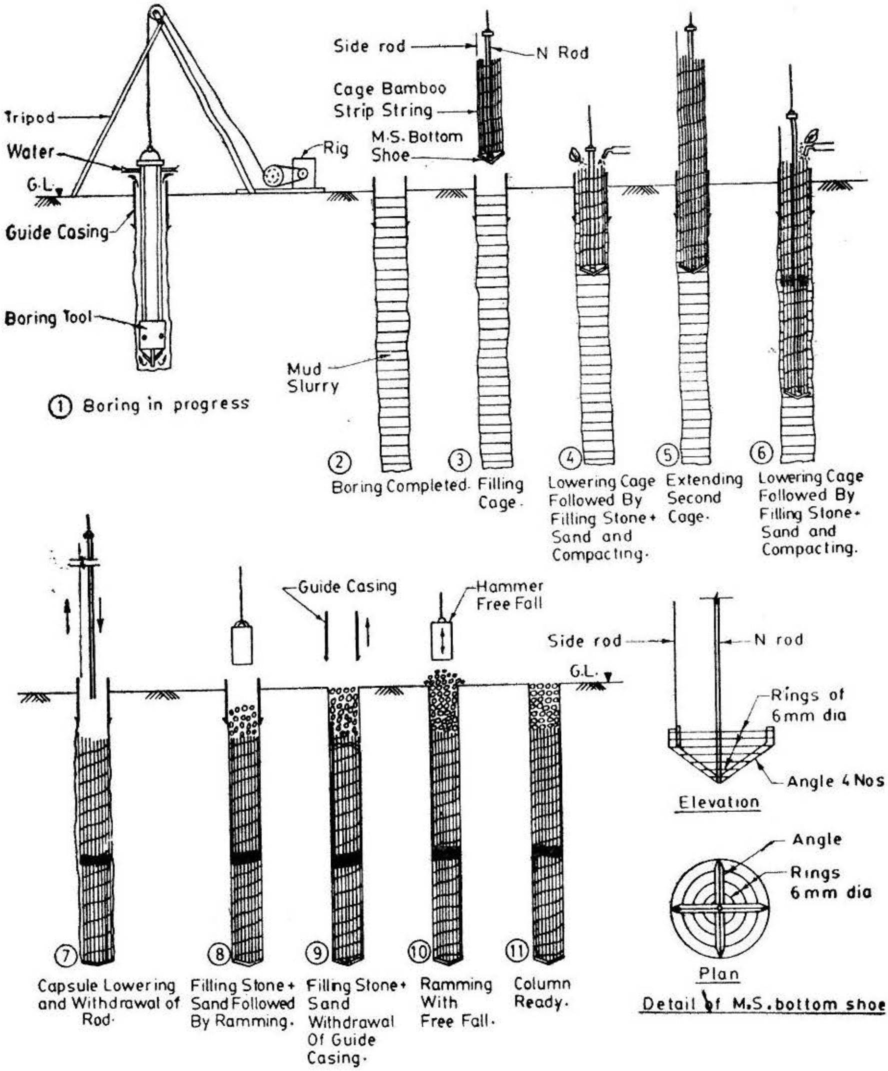


FIGURE 2*b* Installation of stone column through uncased bore hole over ground placement with special shoe

of the yield stress and apply elastic theory to settlement analysis of stone column by considering the stone column to behave like a compressible pile. However, the validity of theoretical analysis is several restricted by the influence of construction operations on the state of stress in the soil mass in the vicinity of the stone columns as well as the strength and the deformation characteristics of the soil. This has resulted in an element of uncertainty in the analysis which has been a cause of skepticism among the theoreticians. An impression seems to prevail that the design approach to stone columns is highly empirical and therefore, a rational evaluation of design and construction alternative is not possible.

At the outset, it should be noted that the empirical element in the design of stone column systems is not significantly different from pile foundations particularly if we consider the piling practice a few years ago. Since it is difficult to estimate the state of insitu stress and the soil characteristics in the vicinity of the stone column, this theory can best be used to evaluate the relative importance of various factors influencing stone column behaviour. Rational extrapolations can then be made from results of load tests and past experience and a reasonable and conservative estimate of settlement and yield load can be obtained.

Estimation of yield load

The cavity expansion theory (Vesic 1972) constitutes the main theoretical basis of estimation of the yield stress or the maximum vertical stress in the stone column beyond which excessive deformations would occur. The cavity expansion theory can be applied to arrive at the vertical yield stress according to the following equation:

$$\sigma_3 = CF + q\, F'q$$

$$\sigma_1 = \sigma_3\, N_\phi \quad \text{where } N_\phi = \frac{1+\sin\phi}{1-\sin\phi} \qquad \ldots(1)$$

Parameters $F'c$ and $F'q$ used in the above equations are subject to several uncertainties. In the first instance, it is difficult to estimate the insitu undrained strength of the cohesive soil in the vicinity of the stone column due to the influence of the installation procedure on the soil characteristics. Initially there is the remoulding of soil during boring and the extent of strength reduction due to this remoulding depends on the method of boring and the equipment used. Recompaction of the soil takes place during the compaction of the granular fill. Again, the extent of recompaction zone and the strength gain depends on the installation techniques- Subsequent to the installation, dissipation of pore pressure occurs and this would also contribute significantly to strength gain.

It must be borne in mind that the extent of strength reduction by remoulding as well as the subsequent gain of strength by compaction or consolidation varies in a radial direction. Significant changes in strength would occur in an annulus of about $2D$ where 'D' is the diameter of the compacted stone column material. Since strength characteristics vary rapidly in the annulus around the stone column, it is difficult to assign a representative value to the rigidity index, which is essential for estimating the cavity expansion factor $F'c$ and $F'q$.

The second element of uncertainty relates to the level of mean stress around the stone column which depends not only on the installation procedure but also the initial strength of the clay. While the capacity of the equipment used for ramming or vibration would determine the extent to which vertical stress can be induced in the stone column during compaction; the actual stress developed would be limited by the lateral resistance of the soil since otherwise the clay would yield and the granular fill would keep on bulging indefinitely.

The third element of uncertainty is with regard to the sectional area and as placed gradation of the stone column material. This obviously depends on the extent of segregation occurring during placement as also by the extent of contamination by local clay. On projects where I was associated, I have preferred a gap graded mixture of medium sand and large size stone aggregate. In this type of granular filling, the sand works into the voids in the stone and finally a dense mixture is formed where all the void space in the stone is filled by the sand. The volume of stone and sand can be measured and if this is used as a field control criteria, it should be possible to make a reasonable estimate of the cross sectional area of the compacted granular fill constituting the stone column at various elevations.

Considering the interaction of all the above complex factors, I found it expedient to use a composite parameter $F'Sc$ which would relate the vertical yield stress σ_v to the insitu undrained strength Cu of the clay. In principle, $F'Sc$ should be equal to $F'cN\phi$ but in practice the strength of the clay insitu at the end of the installation would be quite different and generally higher than the undrained insitu strength C_u. A practical approach to the estimation of σ_v yield would, therefore, be to evaluate $F'S_c$ by conducting load tests and examining the σv values interpreted value of σ_v in relation to the profile of Cu prior to installation. The cavity expansion theory can then be used to make a suitable allowance for increase of σ_v due to the stress changes in the soil mass occurring after installation and during service.

In practice, the contribution of the parameter $F'q$ is found to be small particularly in the critical upper portion within the first few meters of the ground level, Therefore, the design approach could very well be based on consideration of a direct empirical relation between yield value of σv and Cu. I found that this approach is substantiated by Mitchel in his general report (Mitchell 1981). Mitchell does not, however, seem to have considered ihe jnfluence of installation methods. For rammed columns, it should be possible to realise values of $F'Sc$ very much higher than the figure of 25 suggested by Mitchell.

From a practical point of view, a close control on the compaction, quality and the volume consumption of granular material is essential and of vital importance. The emphasis thus shifts from theory to evolving a specification for field control and exercising the field control so that consistent performance can be attained. Design $F'Sc$ value can best be determined by interpretation of load test data on stone column installed according to the stipulated procedure. The development of a control procedure which relies on the 'set' of the rammer and relating this 'set' and consumption requirements for the granular column to the design $F'Sc$ value, thus becomes the basis of a practical design approach. There was no other way of getting around the uncertainties in application of the cavity expansion theory. In Annexure 4, a typical calculation is presented of the analysis of load transfer in a single column load test and derivation of field value of $F'Sc$ from load test data.

From the above discussions it would be evident that the cavity expansion theory is a very useful tool for understanding the factors influencing the yield values of the vertical stress in the stone column and for interpreting load test data so that the test results can be used for evaluating design parameters. At the same time, it must be recognised that the cavity expansion theory cannot and should not be directly used to estimate values of σ_v yield.

In the context of the vertical load carrying capacity of the column, it is necessary to make a critical assessment of the approach to selection of factors of safety for estimation of the working load of stone columns. When the construction is closely controlled, it should be reasonable to adopt a factor of safety of 2 with reference to *F'Sc* values estimated from load test data. When settlements are not critical, it should be permissible to reduce the factor of safety to 1.5 by taking advantage of the inherent capacity of the stone column to redistribute the load since collapse would not occur when yield load is exceeded and the stone column continues to deform without loss of resistance.

Settlement Analysis

Application of theory to analysis of stone column settlement is subject to several uncertainties viz.,

—The method of installation governs the stress deformation behaviour of the compacted granular material as well as the surrounding soil.

—The state of stress is radically modified by the construction operations i.e., boring followed by compaction.

—The strength characteristic of the soil insitu are characterised by heterogeneity and anistropy. This is further accentuated by remoulding which may completely destroy the natural structure. Compaction may restore a part of the strength lost by remoulding. The net consequence of this process is a large variation of strength along the radial direction from the soil-stone column interface.

The value of the stress in the soil developed during installation and the residual stress after pore pressure dissipation varies with the distance from the soil-stone column interface. There is probably an upper limit on the radial stress depending on the existing overburden stresses and the insitu undisturbed shear strength, since the stress during compaction cannot exceed a limiting value; otherwise local yielding would take place.

Evidently, it is extremely difficult to make an evaluation of the design parameters in view of the complex interactions of various factors mentioned above.

In the suggested approach, theory would be used primarily to identify the factors influencing settlement and the relative importance of various factors so that a meaningful extrapolation of the test data can be made and test results are applied only to conditions which are essentially similar.

The problems of settlement analysis can be grouped into the following categories:

(*i*) Situations where the load shared by the soil surrounding to the stone column could be be ignored and the stone column system can be analysed like a group of compressible piles taking the entire load.

(*ii*) Situations where strain compatibility is achieved since the soil is preloaded and the soil-stone column system can be treated as a composite material.

(*iii*) Slip between the stone column and soil is unavoidable especially in the upper zones.

It has been found from the experience that almost all the practical cases fall in one of the above categories.

The first category pertains to field conditions where preloading is not practical and the upper layers of soil are soft and highly compressible. In this category, it is the load carrying capacity which is the critical factor.

The second category wherein preloading prior to installation of stone-column is required is a very economical system; but its practical application needs careful scheduling. The soil may share only 20-30 per cent of the load and the effective preload may only be about 4-6 T/m^2 when the designed loads are of the order of 15-20 T/m^2. This system is well adapted to cases where zones of high applied stress intensity are localised (when preload tends to become ineffective).

In the third category are included structures placed over embankments where structural loads are small relative to embankment load plus drag forces imposed on the stone columns during consolidation under the embankment load. In this case the possible minor settlement of structural foundations can be compensated by placing a small surcharge. The consolidation of the soil in the vicinity of the stone column treated ground would be rapid; hence the time required for placement and removal of the surchargs load may be of consequence (Figure 3).

Methodology of settlement analysis

It would be evident from the aboue discussions that a precise analysis of stone column settlement is generally not required in practice, and practical problems of settlement analysis reduce to only two cases:

(1) When the stone column acts like a compressible pile and the upper layers of soil slip relative to the stone column.

(2) When displacement compatibility is assured by preloading in soft soils and in the stiff soils for the lower layers of case 1.

In the first case, the main purpose of the settlement analysis is to estimate the depth of the layer in which slip occurs. The stone column would gather load progressively from the top unto the lower boundary of the slip zone which coincides with a neutral plane below which the stone column and soil deformations are compatible. The purpose of analysis is to estimate the settlement of the stone column in the upper section, where the vertical stress in the stone column is variable, and to estimate approximately the magnitude of the mean confining stress in the soil. This is required for estimating increase of the load carrying capacity of the stone column due to the added confining stress. In this case, an approximate analysis is sufficient since only the upper and lower bounds of the elevation of the neutral plane are required to be determined. The settlement values can then be established by considering an equivalent uniformly stressed column with reference to the dimensions of the zones sobject to slippage between the stone column and the soil in the top zone of drag and the bottom zone of positive skin friction mobilisation.

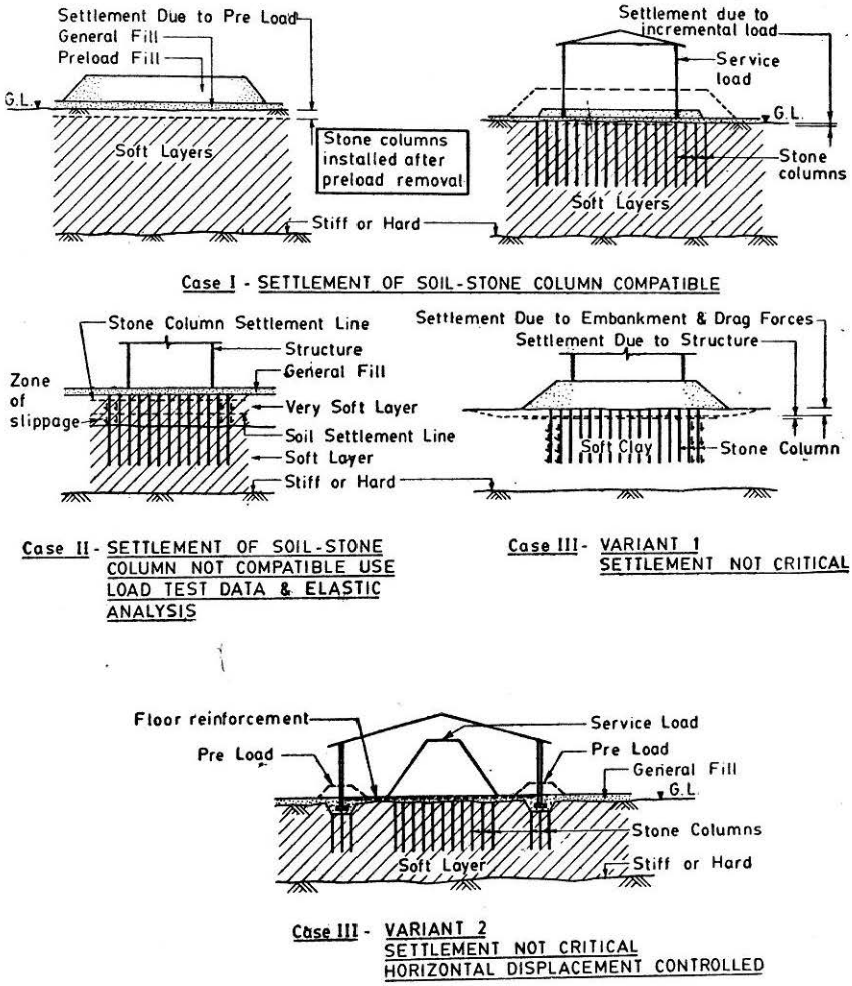


FIGURE 3 Stone column settlement phenomena various cases

In the second case where displacement compatibility is assumed, simple theory of composite materials can be used to estimate stone column settlement.

It would be thus seen that the only parameters needed for settlement analysis are coefficients of compressibility of the stone column and the soil. While routine procedures are adequate for estimation of soil compressibility, values of stone column compressibility can best be established by interpretation of the load test data. Elastic theory can be used with advantage to interprete load test data and to evaluate the compressibility of the stone column. Examples of the analytical processes for interpretation of load test and computation of settlement are presented in Annexure 4

Optimisation of the design

Considering the difficulties of procuring vibratory compaction equipment and the high energy input for compaction of rammed columns, the design approach for Indian conditions should be based on using stone columns of relatively short length, say upto 8 or 10 meters. Stone column treatment should also be restricted, as far as possible to narrow strips in the peripheral areas or otherwise in heavily loaded portions of the structure. This can be achieved in practice by using the stone column in combination with preloading and using structural concepts which make it possible to limit the zone of the ground improvement by stone column to shallow upper zones of the soil. Soil reinforcement in the granular fill above the stone column or in the embankment will also help to reduce stone column depths. The reinforcement could counteract lateral forces so that the stone column would be subjected only to vertical loads and their capacity would, therefore, be increased. Alternatively RCC friction slabs can also be used.

The following concepts have a good potential for further cost reduction in foundation.

(*i*) Experience has shown that the most valuerable part of the stone column is the top 2-3 meters portion. As alternative is being explored (Figure 4) which involves use of lime flyash established soil reinforced by spirals or loops of polymer. The design approach will be explained latter.

(*ii*) Use of soil reinforcement in the sand mat above the stone columns to achieve better distribution of stresses and to minimise load concentration in stone column treated zone.

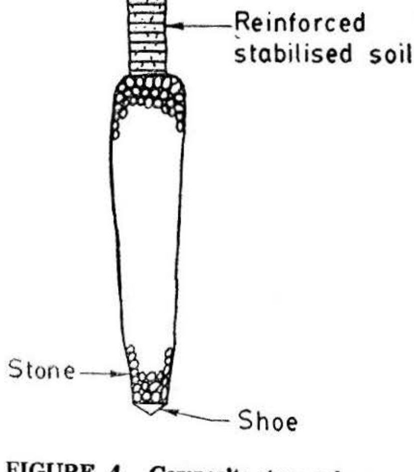


FIGURE 4 Composite stone column

Evaluation of alternative installation techniques for stone column :

The following criteria can be used for evaluation :

—Performance as a measure of ground improvement

—Cost and energy consumption

—Availability of plant

—Ease of field control.

With regard to performance, a simple and direct measure would be the parameter F'Sc which relates the vertical yield stress in the stone column to the underained insitu strength of the natural cohesive strata. Values of *F'Sc* for various techniques are as follows :

Vibrofloatation	Rammed in cased bore holes	Rammed in uncased bore holes
25-30	45-50	40

Evidently, the rammed column installed through cased bore holes scores over other alternative with regard to performance. The rammed stone column installed through uncased bore holes compares favourably with the vibrofloated stone column.

With regard to cost reduction and energy saving, the stone column installed through uncased bore hole is distinctly superior to other techniques as could be seen from Table 4. The quoted prices are very much dependent on the bidding situation, as has been the experience in respect of the 200 mm sand drains and sandwicks. Competition has resulted in bringing down the quoted rates and the present rates reflect the basic cost better than the situation prevailing a few years ago. The vibrofloated stone column is somewhat more expensive in terms of cost and consumes a great lot of energy. The technique for installation of the stone column through the uncased bore hole has a potential for development and eventually costs are expected to be significantly lower than the present estimates which are based on a trial on a small scale.

Equipment commonly used for bored and driven piles can be used for rammed stone columns. Light bored piling equipment e.g., direct mud circulation equipment with 25-40 HP winches can be used for rammed column with uncased bore holes. Although rammed columns are also installed through cased holes with bailer boring, this method is slow and somewhat difficult to control in the field due to the hazard of ingress of clay into the hole through gaps in the casing or due to sand blow. The control is easier with rammed columns installed through cased bore holes with dispensable shoe, but heovier pile driving rigs are required.

It seems to be possible to simplify field control by reducing the stages for compaction to one or two by using heavy rammers and by using a reinforced establised soil column as explained elsewhere.

TABLE 4

Comparative Cost for Stone Columns Installed by Various Methods

1. Stone column type	Vibrofloat	Pre-3 assembled	Rammed
2. Capacity (Ultimate) Tonnes	16	16	21.25
3. Equipment H.P.	250	40	50
4. Equipment cost Rs.	15,00,000	2,00,000	2,00,000
5. Output Hours/per stone column (10 M depth)	1,25	4 to 6	6 to 8
6. Equipment cost/hour in Rs.*	375	50	60
7. Fuel cost (effective)/ Hour in Rs.	120	12	15
8. Labour/Hour in Rs.	40	25	25
9. Cost/hour Total 6, 7, 8,	535	87	100
10. Cost per stone column Rs. (other than material)	668.75	348 to 522	600 to 800
11. Material Cost Rs.	663.0	510.0	590.0
Total Rs.	1331.75	858 1032	1190 1390
12. Estimated quoted rate Rs.	1790	1100 1300	1500 1750
13. Cost per unit capacity Rs.	106	69 to 81	94 to 109
14. Energy per tonne capacity in Ltr. Diesel	6.2	2.9	3.0

Soil improvement by deep insitu mixing methods

Mixing of admixture such as lime and cement with soft and loose soil to form columns ; piers and walls insitu is a recent development which has been applied extensivley in Europe and Japan. (Broms and Boman, 1976, 1979a, 1979b, Okumura and Terashi 1975, Pilot 1977, Sokolovic et al 1973 and Terashi et al 1979). Insitu mixing is achieved by introducing the admixtures into the subsoil through a rotary drill equipped with special bits to advance the hole as well as mix the soil. Here again thorugh mixing is of vital importance. Much of the success of the new insitu mixing methods is due to the excellent mixing capabilities of the equipments.

Cement is more effective for treatment of granular and fine grained soils for law plasticity where the cement contents are usually of the order of 3 to 10% of the dry weight of the soil.

Addition of the lime contributes to improvement of soil properties in several ways. In the first instance, there is the reaction of hydration which occurs with quick lime as well as slaked lime. This contributes to increase of strength and reduction of permeability. Quick lime has the further advantage that the heat of hydration results in reduction of the water content. The consequent expansion also helps to compress the soil in the vicinity of the lime column.

Quick lime is, however, a difficult material to handle and special equipment is required consisting of air tight containers and pneumatic conveyors working with dehumidifined and compressed air. There is the further problem of storage of the lime and its transportation. This requires special packing land handling facilities.

Equipments for deep mixing of quick lime have to be imported. The utilisation factor would, therefore; be low and breakdowns would be frequent when the population of the equipment is small. Due to all the above constraints, the potential for applications of these techniques in India is very limited. Although simple equipment can be used for mixing of cement, the treatment would be expensive due to the relatively high cost of cement.

I initiated a trial of an alternative technique where quick lime and sand were assembled in a capsule and introduced into the bore hole through casings provided with dispensable shoes. Equipment used for installation of sand drains by displacement method was used. Field trials were not successful due to the difficulties of handling the quick lime and the ingress of moisture into the driven tubes.

All the above techniques are energy intensive in respect of each of the cost factors such as materials (cement), equipment, horse-power, facilities for handling of materials at site. Quick lime is energy intensive if the cost of transport, handling, packaging and storage are included. A low cost alternative exists viz., of the installing columns and piers of lime flyash stabilised soil with reinforcement of strips of polymers. Energy saving in this alternative approach would be very substantial and the estimated costs compare very favourably with the alternative of deep mixing of quick lime. Low cost equipment such as trenching and boring eqipment with bentonite (light bailers, DMC chisels, and the tools developed by CBRI) commonly available with Indian contractors can be employed for the installation of columns and piers. This alternative will be further detailed.

Compaction of deep soil layers by tamping with heavy weights and vibro-compaction

Compaction of deep soil layers may be required for control of settlement or prevention of liquefaction. Techniques commonly used for deep compaction such as vibro compaction, compaction piles and tamping with heavy weights are energy intensive. The input of energy increases with depths and in case of heavy tamping, the energy input increases according to square of depth. Estimated values of energy input for heavy tamping equipment for compaction soil layers of various depths are indicated below :

Depth of Soil layer M	Value of Product of weight and fall			Weight Tonnes	Fall Meters
	As per Menard	Probable Upper limit	Mean Value		
3	9	36	22.5	8	8-12
4	16	64	40.0	8	5-8
5	25	100	62.0	5	5·7

Considering that 5 ton equipment can be conveniently mobilised under normal conditions in India, the economic limit of compaction by heavy weight would about 4 meters. For greater depths, the energy consumption could be excessive and the process may become uneconomical.

For prevention of liquefaction, it is possible to use piles since it has been established by theoretical studies that liquefaction is prevented in an area within a distance of about 2-3 diameters from the periphery of the pile group (Yamanouchi et la 1978). Ordinary installation of piles for this purpose would be expensive and the piles cannot be conveniently used in combination with dynamic compaction methods since the dynamic compaction by tamping with heavy weight may damage the piles. A possibility exists of developing a low cost pile of reinforced low modulus concrete reinforced by polymer rings. The cost of the reinforced material would not exceed Rs. 120/m^3. It would be seen later, that a stress as high as 15 kg/cm^2 can be withstood by such piles. Dynamic stresses during impact of even 2-3 times the allowable static stress may be permitted in polymer reinforced piles of low modulus mortar or concrete.

A combination of tamping by heavy weights of the shallow surface layers of ground provided with reinforced cement flyash concrete piles appears to be a very attractive proposition. The cost of heavy tamping and the energy requirement would be quite moderate if dynamic compaction technique is used for compacting only the upper 3 to 4 meters of the loose soil deposit. When pockets of silt and clay are present, stone columns may be provided and these could be interspaced with the reinforced low modulus piles in order to optimise the system. The stone columns, apart from compacting and reinforcing the soil, will also provide drainage paths which should enhance the efficacy of the system for prevention of liquefection.

Annexure 3

Effect of well resistance on design of Sand Drains

Classical solutions for design of sand drains assume an ideal drain of infinite permeability or in other words, drain is assumed to have infinite capacity to drain all water which is squeezed out of sand drain influence area. Barron (1948) for the first time brought out theoretical formulations which take into account the smear effect and well resistance for estimate of per cent consolidation. Richart (1957) also considered the well resistance

Annexure 1

Cost and energy analysis for vertical Drains

Input for Treating 100 m² Area

Degree of consolidation	90 per cent	Site A
Time of consolidation	90 days	Ch=10 m²/year

Cost component		Drain diameter in mm			
		400 ϕ	200 ϕ	65 ϕ	150 mm wide fabric
Fuel consumption	Ltrs.	628*	317	450	583
Machinery deprecia—tion	Rs.	3,015	724	1,200	3,129
Material cost (Sand, bamboo, PVC tubing/ HDPE Sheath	Rs.	1,391	2,960	3,921	15,000
Transportation (mainly sand with 25 km. lead)	T-km.	668	364	60	400

Equivalent Energy for Treating 100 m² Area in Litres Diesel

Cost component		Drain diameter in mm			
		400 ϕ	200 ϕ	65 ϕ	150 mm wide fabric
Fuel consumption	Ltrs.	628*	317	450	583
Machinery deprecia-tion	Ltrs.	503	120	200	520
Material	Ltrs.	——	348	571	2,500
Transport	Ltrs.	33	18	3,0	20
Sub-total in Ltrs.		1,164	803	1,224	3,623

*After making suitable allowance for low utilisation of engine power.

Annexure 2

Cost and energy analysis for vertical drains

Input for Treating 100 m² Area

Degree of consolidation 90 per cent
Time of consolidation 90 days

Site B — $Ch = 1$ m²/years

Cost component		Drain diameter in mm			
		400 φ	*200 φ*	*65 φ*	*150 mm wide fabric*
Fuel consumption	Ltrs.	1,406	206.5	173	346
Machinery depreciation	Rs.	6,750	944	461	1,950
Material cost (Sand, bamboo, PVC tubing/ HDPE Sheath)	Rs.	3,114	6,827	1,936	42,126
Tranportation (mainly sand with 25 Km. lead)	T-Km.	1,495	504	32	115

Equivalent Energy for Treating 100 m² Area in Litres Diesel

Cost component		Drain diameter in mm			
		400 φ	200 φ	65 φ	150 mm wide fabric
Fuel consumption	Ltrs.	1,406	206.5	173	346
Machinery depreciation	Ltrs.	1,125	78.5	77	496
Material	Ltrs.	—	295	320	2,380
Transport	T-Km.	75	13	2	9
Total in Ltrs.		2,606	593	572	3,231

and concluded that well resistance will not be significant for a=7-15, with well diameter of 300 mm. In the situations then prevailing, well resistance was not critical since large diameter (300 mm to 400 mm) drains were used in layered situations in U.S.A. while cardboard drains were used in Sweden, for homogeneous soft clays of low permeability (Kjellman, 1948).

It has been established that the effect of well diameter on the time of consolidation is not as pronounced as that of well spacing, and with the decrease in diameter of a drain well the efficiency of drain is not proportionately reduced. The obvious development was tendency to go for small diameter drains (Hanso, 1979) and (Mitchell, 1981). However, with the introduction of these small diameter drains, evaluation of smear effect and well resistance became critical. Although evaluation of smear effect is a routine matter after Richart (1957) introduced the design charts, evaluation of well resistance is still an involved procedure. Design procedure is now available to check whether well resistance is critical and needs evaluation (Bhide 1979). Attkinson and Eldred (1981) developed a computer programme to evaluate the effect of smear and well resistance.

A number of sand drain test plots have been recently constructed which are elaborate in 'Third Geotechnique Symposium on vertical drains' (in print). (Geotechnique, March 1981). In the opinion of many authors in this Symposium (Davis et al 1981), the performance of fabric drains is not as goods as sand wicks or 200 mm dia. drains. Although no definite conclusions have been drawn as to the reasons for slightly lower efficiency of fabric drains, it is believed that well resistance and smear effects do affect the performance of small size drains. Computation of fabric drain spacings for highly anisotropic soil with Ch of order of 10 m^2/year and $\frac{C_h}{C_u}$ Ratio=10 may be unrealistic. Field verification of performance is necessary before arriving at any conclusion of cost. In this respect, the manufacturers' claims of high drainability values should be considered with a caution. In fact, in layered soils, the fabric drain does not work out to be economical under Indian situation. This is mainly due to very small spacing, which becomes necessary, when smear effect and well resistance are properly accounted for.

This means that drainability of small diameter drain is very important. It also brings out one important fact, that if these drains are to be introduced in India, only high technology backed up companies can provide necessary knowhow for production of these drains especially with regard to permeability and filter characteristics.

Annexure 4

Analysis of Stone Column Load Test Data

With regard to the yield stress the critical portion of stone column is usually at a shallow depth below the ground level. Load tests should be

Stress in Stone Column in Load Tests

Col. No.	Nominal diameter mm	Material stone : sand	σ_v t/m^2
137	750	5:1	67.6
104C	750	5:1	72.6
38C	750	5:1	46.0
23	750	5:1	63.6

carried out providing a device which would transfer the load to a level as close to the critical level as possible.

Assuming that there is no load transfer due to skin friction, the vertical stress developed in typical stone column are shown in the Table (p. 29).

The above stresses correspond to 'failure' load which is taken as the load on the stone column at which log $\sigma v/C_u$ Vs. ρ/d curve becomes almost vertical, here σv is the vertical stress in the column and C_u the undrained strength of surrounding clay at the corresponding depth, ρ is the observed settlement and d the nominal diameter of the column.

Sectional area of the stone column is calculated on the basis of actual consumption of stone and sand. Stone and sand in the proportion specified for each column is measured loose. Compacted volumn is taken as 0.8 times the loose volume of stone only since with a ratio of stone to sand in the range of 0.2 to 0.4, the sand fills the voids in the stone.

Laboratory experiments on crushed stone 20-80 mm size and medium sand using vibratory compaction have confirmed the validity of such an assumption. For purposes of analysis the column cross section is considered uniform between the depths were 'set' is measure.

Load Transfer and Settlement

Load transfer in the stone column is analysed by considering two separate sections of the stone columns comprising an upper section and a lower section and the load sharing between the two sections is estimated by trial to ensure displacement campatability. Settlement of the soil surrounding the upper part of the stone column is estimated by considering this part as a short compressible floating pile using elastic solutions (Poulos and Davis 1974). The displacement of the lower part of the stone column is estimated by treating the stone column as a compression member connecting the upper part to the lower end bearing zone which transfers the load to the soil by combination of end bearing and skin friction. For estimation of settlement of this end bearing elastic solutions can again be used.

A trial value for the stone column E value generally of the order of 5000 t/m^2 can be assumed on the basis of experience, which is within the range of values mentioned by Mitchel (1981). However, an element of uncertainty always exists regarding the estimation of undrained modulus for soil, several trials may therefore be required before a compatible value is found. It should be realised that the purpose of settlement analysis is to check the order of magnitude of anticipated settlement of stone column soil system and a high order of accuracy is not necessary.

Settlement Analysis

It is possible that a netural plane develops below which the stone column and soil settlements will be compatible (Figure 4).

For settlement computation an estimate of the equivalent length is required which can be approximated by considering the drag forces and the relative compressibility of the soil layers and the stone column compressibility.

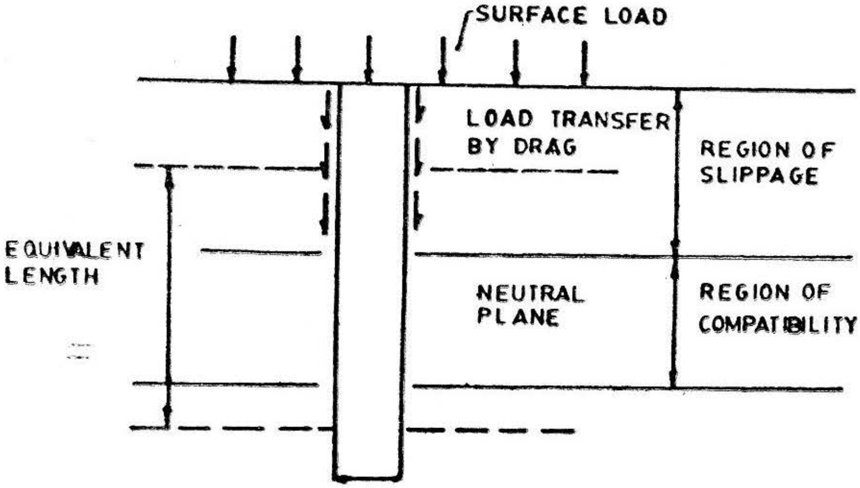


FIGURE 4

Soil Reinforcement

Background

Several thousand reinforced earth structures have been built in Europe and U.S.A. Since the first application of this technique in France by Vidal in 1966. However, the reinforced earth technique has not been introduced in India so far; the main constraining factor being the availability and cost of reinforcing materials. An impression seems to prevail that the savings are not sufficient to justify importation of materials and technology and an element of risk is involved if the technique is to be used for major structures. I, therefore, thought that a practical approach would be to apply soil reinforcement methods to structures which do not constitute a safety hazard. Further if constraints regarding availability of cement and steel can be overcome by use of soil reinforcement, project authorities can be persuaded to use low cost materials of limited working life for structures which can be replaced at intervals of 10-15 years.

I realised at the outset that the potential for application of soil reinforcement methods cannot be realised unless cost of reinforcement is brought down and low cost facings are developed. It is also necessary to routinise design procedures and construction methods. The scope for application would be further extended if soil reinforcement can be used for cohesive soils and expansive clays. This is possible only if system of reinforcement is such that its performance does not depend on the adhesion between soil and reinforcement. This implies that the French concept of 'reinforced earth' cannot be applied directly and some type of anchorage must be used for the soil reinforcing strips. Alternatively, the reinforcement can be in the form of loops so that each soil element is confined within a loop. The reinforced soil mass may then be considered to be composed of a number of interlocking reinforced soil cells with each cell behaving somewhat like a cellular cofferdam.

Suggested system of soil reinforcement

Two alternative reinforcement systems are proposed as described below :

In the first alternative, the reinforcement will be in the form of a series of loops. Blocks of stabilised soil with inserts of brick or clay tile, placed within the loop, will constitute a series of anchorage (Figures 5 and 6).

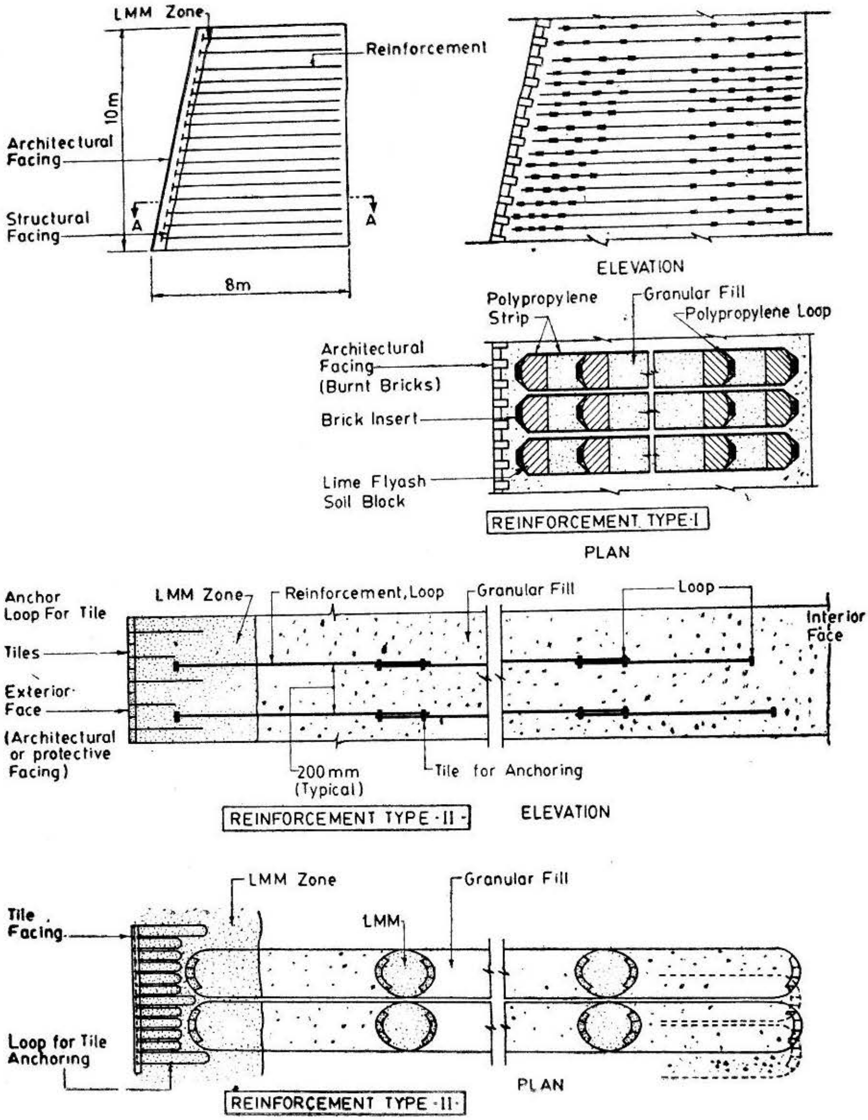


FIGURE 5 Reinforcement system

The mechanism of shear transfer from these anchorages involves passive resistance of soil as well as friction. Test results on bar mats and rubber tyre rings used as soil reinforcement lend support to the anticipated efficacy of the proposed system. (Forsyth 1978, Cartier, Long et al 1981).

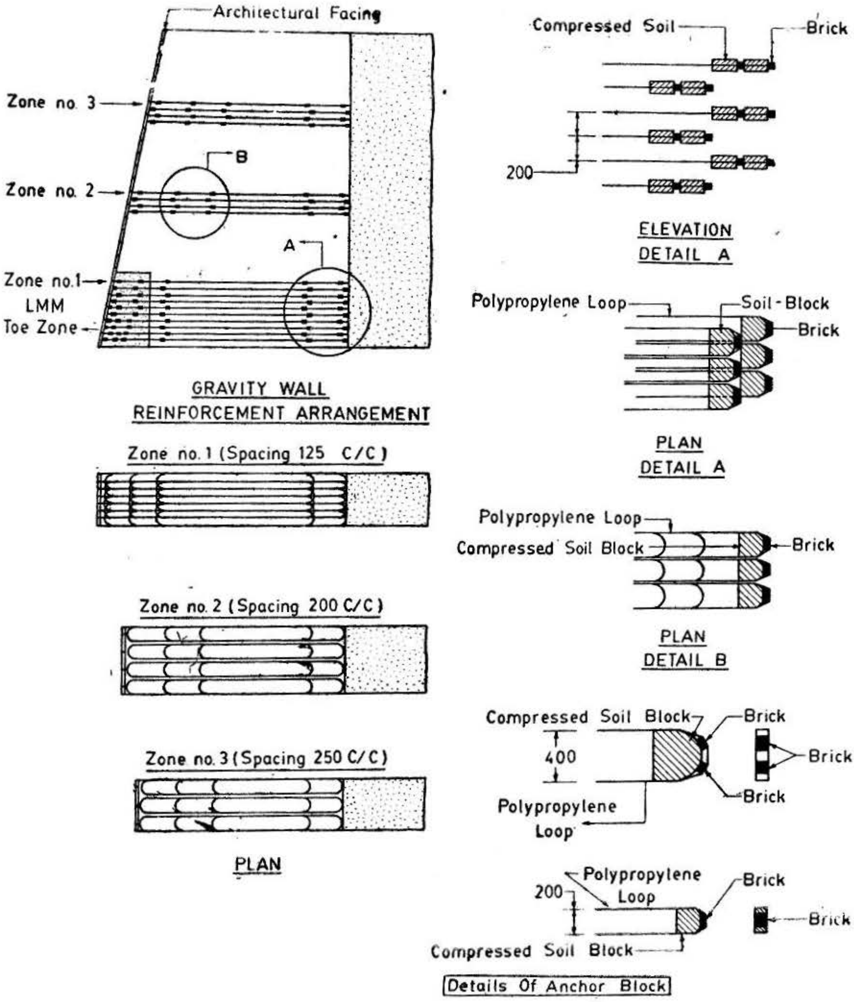


FIGURE 6 Details of type 1 reinforcement for a gravity wall

One set of inserts will be placed in the 'active' zone while another set will be placed in the 'reactive' zone. It should be possible to reduce the length of the reinforcement in this system by taking advantage of the improved shear transfer capacity. However, current practice regarding length of reinforcement can be used as a provisional and conservative basis of design.

In the second alternative, the reinforcement would be in the form or rings and loops. When the soil is subjected to vertical stress, the reinforcement would restrain lateral deformation and confining forces would be mobilised. Test data from triaxial test can be used to analyse the behaviour of the soil confined by reinforcing rings and loops. The shear on horizontal planes would, however, be transferred through the soil. The soil should, therefore, have sufficient shear resistance to satisfy requirements of internal stability. This could be achieved in practice by control of initial moisture during placement and by a combination of imperevious zones, filters and drains. Performance of the soil reinforcement in this system would not depend on adhesion and friction between the reinforcement and soil. Reinforcing materials having a smooth surface can, therefore, be used. This makes it possible to employ commercially available strips of polymers and low cost natural organic material such as strips of bamboo.

Field applications will be limited in the first instance to the type I reinforcement. Design principles for reinforced earth system currently in use can be adapted to the new system. A research programme has been initiated to study the behaviour of the second type of reinforcement. Routinisation of design procedures and detailing of type II rainforcement must await the completion of the experimental and analytical work.

The proposed systems of reinforcement are illustrated in Figure 5 and details of type I reinforcement system for a gravity wall are shown in Figure 6. The following distinctive features of the system should be noted.

—There is an internal zone which constitutes the structural facing which is quite distinct from the architectural or external facing. In the first system of rainforcement, an array of the blocks of lime flyash stabilised soil placed with in the loops constitute such a facing. Generally two or more rows of blocks will be required to form the facing. Alternatively, a layer or zone of low modulus material consisting of lime flyash stabilised soil may be used in the second type of reinforcement.

—The external facing is flexible and a slip zone is provided between the external facing and the structural facing. The facing elements themselves will have a 'break' where movement would be permitted. The architectural and protective facing would not, therefore, attract the drag forces due to differential settlement of the soil and facing. The facing elements can be chosen according to the functional requirements since the structural behaviour of the system does not depend on the type of external facing. Further, local damage to facing would not impair the structural performance of the soil reinforcement.

—Another zone of lime flyash stabilised soil may be provided at the inner boundary of the reinforced soil system. This zone would constitute an impervious element when the reinforced soil is pervious. This impervious zone in conjunction with a drainage system would bring down the pore-pressure in the reinforced soil mass.

—In cohesive soils, especially expansive clays, it may be necessary to encapsulate the reinforced soil mass in order to minimise the moisture variations. This could be achieved at a relatively low cost by providing membranes of HDPE or bitumen impregnated fabrics.

—For hydraulic structures, the internal zone of lime flyash stabilised soil could serve as a transition zone between the unreinforced soil mass and the filter/drainage layer and thus ensure effective control of internal erosion.

Materials for soil reinforcement

With the proposed systems of reinforcement, wide range of alternatives exists for choice of reinforcing materials. A comparative evaluation of various materials is made in Table 5. Cost of mild steel bars and polypropylene are equal but the cost of corrosion protection must be added which makes mild steel more expensive. Bamboo is evidently the most economical material. Energy saving would be enormous if bamboo which is a renewable material can be used.

TABLE 5

Comparative Evaluation of Reinforcing Materials

(a) Characteristics

Characteristics	Bamboo		Polypropylens		Polyethylent		P V.C. rigid
	Poor variety	Good variety	High density	Low density	High density	Low density	
Specific Gravity	—	0.7	0.9	—	0.97	—	1.35-1.50
U.T.S. Kg/cm²	1000	2000	21000	1000	260	150	470-710
Elongation at break	—	—	20	—	500	600	—

(*b*) *Costs*

	Bamboo	Poly-propylene	Mild steel Bars	High Tensile steel
UTS kg/em²	1500	6000	5200	——
Yield kg/cm²	1200	——	3000*	12,000
Allowable stress kg/cm²	600	3000	1400	9,000
Cost/lit. Rs.	3	60	28	60
*Equivalent Sectional area cm²	2.5	0.5	1	0.16
Cost of 1000 cm. length with equivalent sectional area	7.5	30	30	9.6
Cost ratio	0.25	1	1	0.32

*Area of member having a strength equal to a Mild Steel Section with an area of 1 cm².

*Average value for commercial grades.

For H.T. wire, the cost of corrosion protection must be added and there also the hazard of stress corrosion failure.

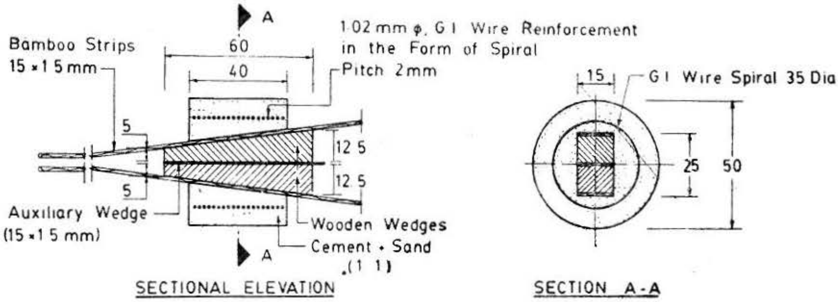
Bomboo as reinforcing material is subject to the hazard of insect, fungus or virus attack and there is also the difficulty of splicing bamboo strips. Known methods of preservation which have been used for wooden poles for rural electrification over the last 25 years such as ASCU treatment can very well be adopted for preservative treatment of bamboo. ASCU is a water soluble preservative and the treatment process for bamboo is very simple. The preservation of bamboo has been extensively studied by F.R.I. In 1978, I introduced bamboo strips for reinforcement of the floor ore of an are yard in Goa. This is believed to be the first application of bamboo strips for soil reinforcement. An evaluation of the performance of ASCU treated strips is in progress. Since the strips were tested for strength characteristics before installation, an indication of the strength deterioration can be obtained by testing the strips which have been subjected to alternative drying and wetting. Due to tidal variation, the hazard of inesct attack was quit servere at this site.

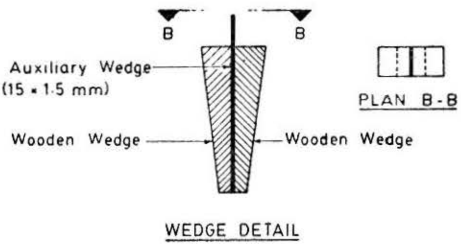
According to the studies by Fang (1981) bamboo strips embedded in concrete have a very long life. There is reason to believe that fungus and organic attack would be inhibited in an alkaline environment. It would be worthwhile to evaluate experience of traditional constructions where fibres were embedded in lime mortar. There are constructions in China which are over 100 years old where composite construction of sun dried bricks and bamboo have been used. A bamboo was placed in holes within the bricks and the annular space around the bamboo was filled with lime mortar. The bamboo members are reported to be unaffected by insect attack. (Fang, 1981).

It is also possible to encase the bamboo reinforcement lime fly ash mortar which in turn could be encapsulated by a suitable film of an impermeable polymer like laminated H.D.P.E. or polypropylene. It could be anticipated in the light of the above observations that long life and durability can be imparted to bamboo reinforcement by developing suitable forms which would consist essentially of an assemblage of bamboo strips encased in cement or lime flyash mortar which may again be confined by spiral of polymer and further encapsulated in polymer sheaths.

Available length of bamboo strips is generally restricted to 3 meters. It was, therefore, necessary to develop designs of splices suitable for field application. After experimenting with various alternative (Datye, Nagaraju 1978 and 1981) two splice designs have been developed. In the first design, a wedge anchor is used in combination with loops. Tests have confirmed joint efficiencies better than 70 per cent and there is a potential for furter improvement (Figure 7). Another splice design has also been developed which involves a winding of a fibre over a lap joint of strips. In both the designs the anchorage and lapping zones should be roughened by applying sand to resin coated bamboo trips. A strip size of 1-1.5 mm thickness and width of 10-20 mm is considered suitable. This also happens to be a common size for bamboo matting material. Froms of reinforcement based on the above two designs of splices are illustrated in Figure 8.

Although results of laboratory test indicate a good potential for use of bamboo strips as reinforcement, it is desirable to develop alternatives with commercially available materials and with a proved performance with regard to durability. Such a design could be used where a longer life is required and also as a material for transitional phase until preservative treated bamboo strips become commercially available.





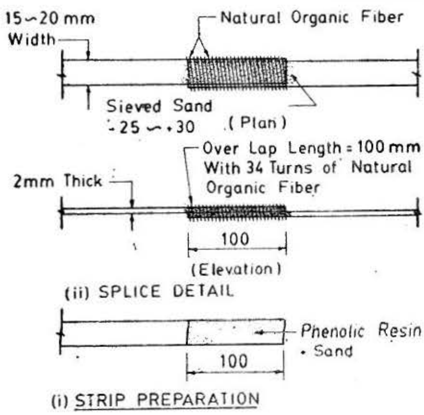
(a) PRECAST ANCHOR

TEST RESULTS - PRECAST ANCHORS

(I) Sr no.
(II) Area of Bamboo Strips (cm^2)
(III) Wedge Slope
(IV) Splitting Tension in Anchor (kg)
(V) Ultimate Designed Capacity (kg)
(VI) Max Load Taken (kg)
(VII) Stress in Anchor (kg/cm^2)
(VIII) Anchor Efficiency (%)

I	II	III	IV	V	VI	VII	VIII
1	0.6	1 8	1000	720	625	1040	86
2	1 8	1.8	3000	2160	1500	830	69

Ultimate Stress in Bamboo = 1200 Kg/cm^2
Allowable Stress in Bamboo = 600 Kg/cm^2



Method .-
(i) Dip Overlap Portion of Bamboo Strip in Phenolic Resin
(ii) Spray Sand & Cure @ 80°c For 1 Hour
(iii) Wind Resin Treated Natural Fibre on The Overlap Portion & Cure

THIN STRIP BAMBOO JOINT

(I) Sr no
(II) Gross Sectional Area (cm^2)
(III) Max Load (kg) in joint
(IV) Ultimate Designed Capacity Of Joint (kg)
(V) Stress (kg/cm^2) in joint
(VI) Efficiency Of Joint (%)

I	II	III	IV	V	VI
1	0.23	260	276	1104	94
2	0.28	332	336	1194	98

(b) THIN STRIP BAMBOO JOINT

FIGURE 7 Anchor and Splice for bamboo

Polypropylene strips

Polypropylene is known to be a durable material and tubing of polypropylene embedded in earth dams for piezometric observations are reported to have a long service life. Recently, polypropylene strips have become available in India for the packaging industry and their costs and characteristics are as follows :

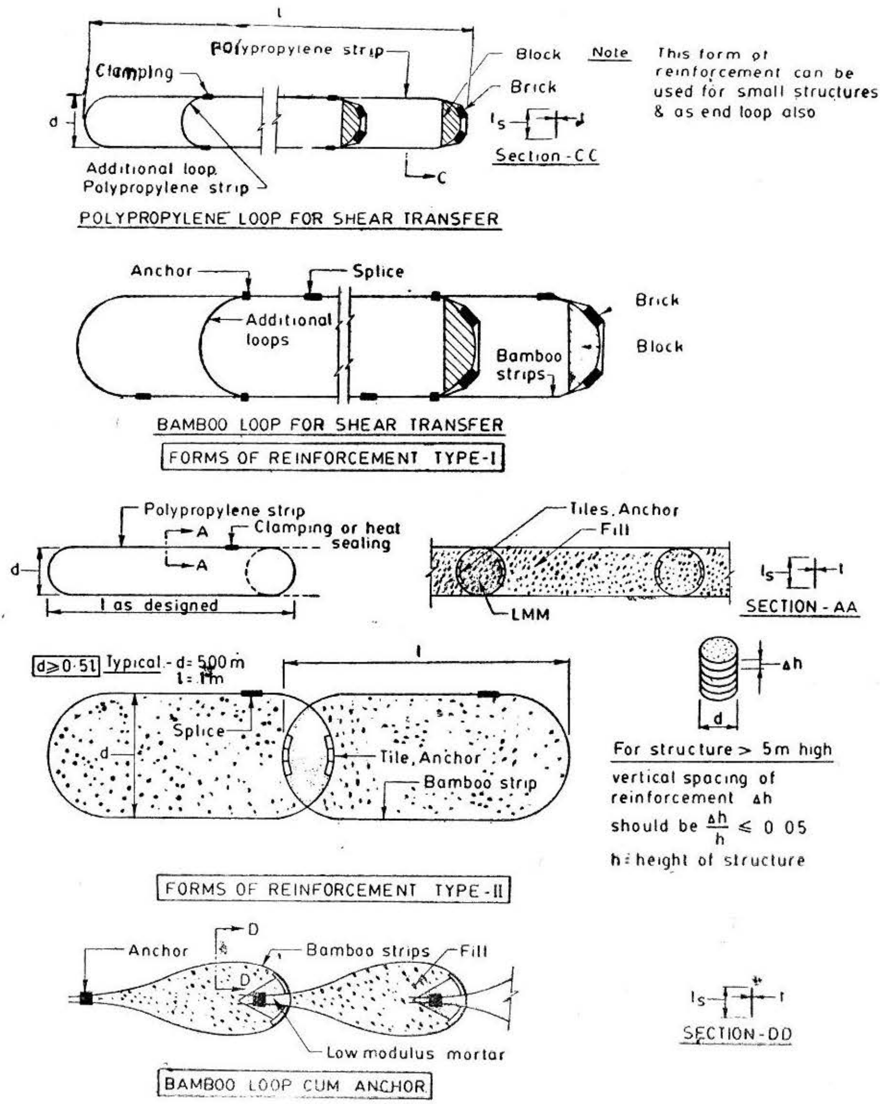


FIGURE 8 Forms of reinforcement

Size : Range—9 mm to 12 mm (width)
0.5 mm to 0.6 mm (thickness)

Ultimate Tensile Strength	:	2100 kg/cm²
Cost/Litre	:	Rs. 22.00
Elongation at break	:	20 per cent (approx.)

Facing

The proposed concept of reinforced earth wall admits of use of variety of facing materials depending on the nature of application (Figure 9). Dry stone masonry facing can be used for a wall with a small batter if stone is available locally at low cost. Alternatively, a dry facing of country tiles or bricks with bands of concrete may be considered.

If fuel for burning country tiles is not available or is unduly expensive, it should be possible to use a facing of compressed lime fly ash stabilised soil with fibre. In fact, this kind of facing seems to have the greatest potential for application.

Use of an anchor or a tile for the facing would reduce the requirement of the thickness in situations where the facings are not subject to forces of water current. Precast and pre-cured compressed stabilised soil blocks with a thin facing of high quality clay tile anchored by reinforcement into the stabillsed soil have also a very good potential for application.

Situation often arises when erosion resistance is a critical factor. Precast concrete, fibre reinforced concrete, or high grade burnt clay tile can be used in such cases.

Field Trial

A trial construction of a small drop structure with facing elements of H.D.P.E. tubes filled with soil and strip of poly-propylene reinforcement was mad recently to test the feasibility of the reinforced earth construction. The experience shows that the construction is simple and the fall has been able to withstand the erosive attack and over-turning action of water.

Another design of fall has now been worked out which is based on facing elements of country tile or burnt bricks. Cost studies indicate that this design sould cost about 50% of the conventional masonry or precast concrete construction. Cement or steel is totally eliminated and transportations will be reduced to a bare minimum while the energy saving will be very significant.

Cost Comparison

Results of a comparative evaluation of the cost of typical gravity walls 5 and 10 mtr. high subject to lateral earth pressure are presented in Table 6 The basis of analysis of the walls and the reinforcement provisions are furnished inAnnexure 4. The unit rates and the relevant analysis is also summarised in Annexure 1 and 2.

The conclusions that emerge from the camparative cast study are that under Indian conditions, the reinforced earth wall with concrete facing and galvanised iron strips has no cost advantage over the conventional cement masonry wall. There is, of course, some saving relating to concerete walls which is not a common material for gravity walls. There is hardly any justification for introduction of the French system of the reinforced earth on the basis of cost. However, in special situations particularly when foundation depths are large for gravity walls, the reinforced earth system may be still favourable. The alternative design based on use of available polymer strips, bricks and stabilised soil blocks shows a cost advantage of almost 50%. This has a very good potential for application since the construction would be carried out with commercially available material and research is needed only to confirm the assumption regarding the shear transfer capacity of the anchor blocks. The saving would be even greater if lime stabilised soil blocks are used in lieu of brick facing and anchor blocks and further polymer strips are substituted by bamboo.

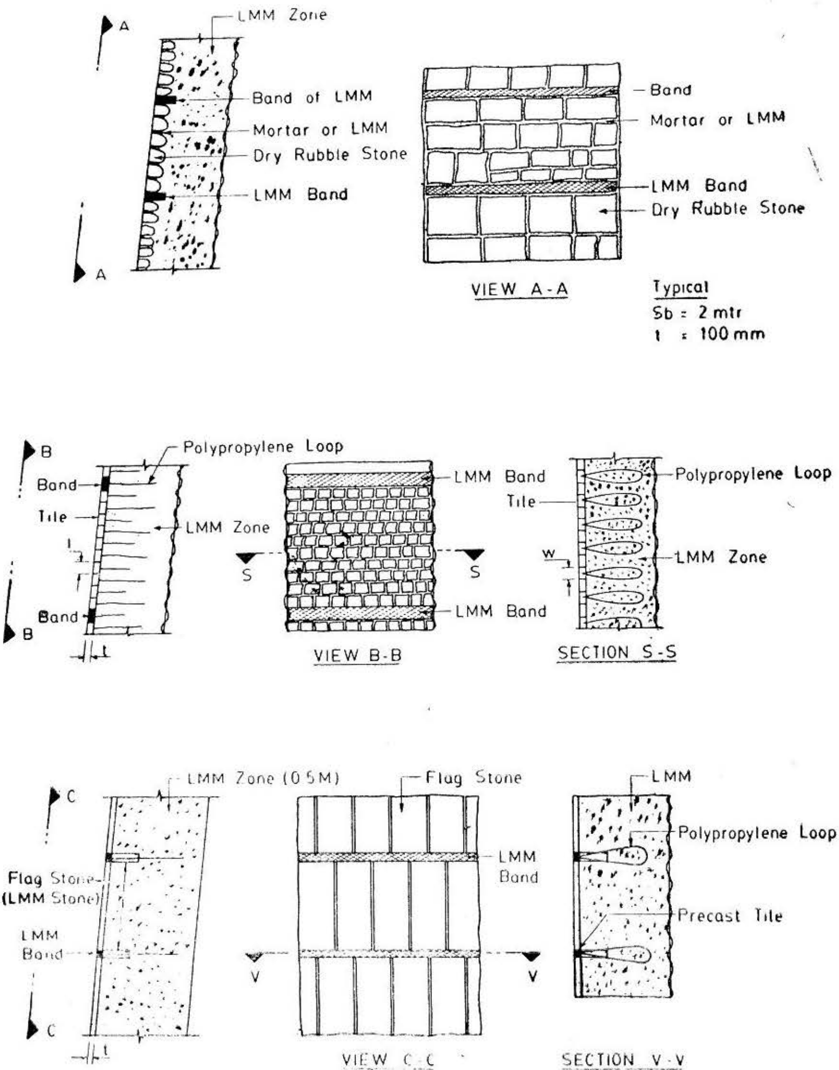


FIGURE 9 Types of facings

Behaviour of reinforced soil

Analysis of reinforced for soil structures involves a complex interaction of the soil and the reinforcement. Methods of analysis used for

TABLE 6

(*a*) Unit Cost/M³ of Wall

Item	Polypropylene loop reinforcement with optimised design & Lime-flyash-stabilised soil block for facing and Anchoraged.		Polypropylene loop reinforcement and burnt brick facing & anchorages of brick & stabilised soil blocks.		Galvanised iron strip reinforcement with R. C. C. facings.	
	10 mt. Height Rs.	5 Mt. Height Rs.	10 mt. Height Rs.	5 mt. Height Rs.	10 mt. Height Rs.	5 mt. Height Rs.
Reinforcement	13.25	6.75	16.00	8.00	30.50	12.00
Anchorin	3.00	5.00	4.00	6.00		
Facing	1.50	3.00	2.00	4,00	18.50	37.00
Misc, Items, Add 5 per cent contigency	0.75	0.75	1.00	1.00		
Unit Rate/m³	18.5	15.5	23.0	19·0	49.0	49.0

(*b*) Comparative Cost of walls of equal length (Consider Masonry Gravity Walls of Height H and Volume WH. Masonry Cost % Rs. 150/m³ Mass Concrete Cost % Rs. 300/m).[3]

Comparative Volume	3WH	3WH	3WH	3WH	3WH	3WH
Relative Cost with reference to Masonry Cost	0.37	0.31	0.46	0.38	0.98	0.98
Relative Cost with reference to Concrete Cost	0.185	0.155	0.23	0.19	0.49	0.49

conventional designs of gravity structures or reinforced concrete structures is not applicable directly due to the non-linear behaviour of the soil which is also very much dependent on the initial stresses and the stress history. The reinforcement may not also behave elastically if it consists of polymers or natural fibres. A further complication arises due to the slippage of the soil relative to the reinforcement, localised yielding and also because the soils in field conditions are anisotropic, non-homogeneous and their behaviour is time-dependent and sometimes the deformation of the reinforcement could also be time-dependent. Stress analysis is further complicated by the interaction between the soil and the reinforcement and the influence of the facing as well as inclusions on the soil mass behaviour. Inspite of these limitations many successful constructions have been carried out in reinforced soil. Even before the recent development initiated by

Vidal came into practice, large number of cellular coffer dams have been designed and constructed and the cellular coffer dam is essentially a reinforced soil mass.

The above difficulties in the analysis can be overcome by basing the design approach on alternative postulations of soil behaviour. The two postulates described below can be considered to be substantiated by past experience and I would like to present the evidence in support of the postulates so that a suitable design approach can be adopted for specific application depending on whether the first or the second postulate is considered. If necessary, the structural geometry, the arrangement and the capacity of the reinforcement can be so chosen that the behaviour is satisfactory according to either of the postulates. Thus, when lhe proposed system does not exactly conform to the reinforcement system used in the past, geometry can be decided conservatively and a reinforcement provision is made which satisfies both the postulates.

Postulate-1

(*a*) Reinforced soil behaves as a quasi cohesive solid which is virtually inextensible in the lateral direction i.e., in the direction of reinforcement.

(*b*) The reinforced material behaves as a linear elastic plastic Mohr Coulomb solid and elastic theory can be used to design structures of reinforced soil by ensuring that the Mohrs Circle for the working stress plots within the Mohr Coulomb envelope for the material with a reasonable margin of safety.

(*c*) The margin of safety in the design for values of parmeters 'C' and 'ϕ' with regard to the Mohr Coulomb envelope for the material should be related to the energy absorbing capacity of the reinforced stabilised soil system. A relatively low value, say 1.5, may be acceptable for ductile reinforcing material and 'plastic' soil which continues to deform after ultimate stress is reached and no sudden collapse occurs at strains considerably higher than the, strain at the ultimate or yield stress. In other words, for a stabilised soil with a brittle behaviour and reinforcing materials of low ductility a high margin of safety is necessary.

Observations on reinforced earth abutments reported by Juran, schlosser et al (1978) also substantiate the above postulation When elastic theory is used for estimating stresses, the relative deformability of the reinforced unreinforced soil mass must be taken into consideration.

If theories of plasticity are to be applied to analysis of reinforced soil behaviour, it is necessary to determine the mode of failure and there is the hazard that conventional theory of plasticity would not be applicable to reinforced soil since the soil behaviour is radically altered by the restraint provided by the reinforcement. Model studies and prototype observations are essential for confirming the theoretical postulation of reinforced soil behaviour. The importance of model studies and prototype observations cannot be overemphasised as could be seen from the controversies that arose a few years ago regarding the French design approach and the approaches based on Coulomb theory. Success of reinforced earth technique in France is largely due to the judicious use of model studies and prototype observations in the Franch development.

At the present state of knowledge, theories of plasticity can be applied only to gravity systems where theories are supported by well organised observational data and extensive model test results. The following postulation of reinforced soil behaviour can be considered to be valid for reinforced soil gravity structures similar to those that have been built and monitored over the last 10 years *i.e.*, granular soils with a system of horizontal reinforcing members.

Postulate-2

A reinforced soil mass with a vertical or a steeply inclined face subjected to gravity forces not behave according to the Rankie or Coulomb theory due to the re straint provided by the reinforcement to lateral deformation. Reinforced soil behaviour conforms to a condition of virtually zero extension in the direction of the reinforcement and the failure of the reinforced mass is, therefore, initiated by break or rupture of the reinforcement for the upper part of the soil which would result in an abrupt increase in the lateral deformation resulting in shear failure of the lower part of the soil mass involving slippage or yielding and plastic deformation of the reinforcement in the lower part of the soil mass. The failure surface can, therefore, be approximated by a vertical plane in the upper part of the soil and curved and inclined slip surface in the lower part. For deformation compatibility, the system should have adequate ductility which is provided by the slippage mechanism since the reinforcement can undergo a large horizontal displacement after slippage even though it may not fail. Better conformity with the theoretical behaviour would be achieved if the reinforcing material is also sufficiently ductile.

Analysis of Reinforced soil structures

It is necessary to bear in mind the difference between classical limit analysis methods generally adopted for design of retaining walls and the observed performance of reinforced earth structures. The restraining effect of the reinforcement which virtually prevents lateral deformation is particularly important in the upper portion of the soil mass. As a consequence, the failure of a reinforced soil corresponds closely to a rigid body movement with thin slip zones. In the upper part of the soil, the thin slip zone can be considered to follow the locus of the points of maximum stress in the re-inforcement and in the lower part of the soil mass it would follow the slip surface for failure analysed by applying theory of plasticity to rigid elastic plastic materials. A careful consideration of the Kinematics is essential and some of the early theories such as those developed by Brinch Hanson based on Kotters equation could be used with advantage.

Alternatively, in order to analyse the stress and deformation fields which develop in reinforced earth structures under normal working conditions which are different from those prevailing at failure, elastic methods of analysis have been used. Finite element method is useful for the study of both model and prototype walls. Both composite body and discrete material approaches are being used. The composite material properties assigned to the continuous elements reflect the properties of the matrix material and the reinforcing members and their composite interaction. For large two dimensional and three dimensional systems, only the composite approach is likely to be economically feasible.

Analysis plays an additional role in the evaluation of the overall stability of reinforced earth systems. In this respect, conventional design methods for gravity walls can be used for reinforced earth structure. It is, however, necessary to take into consideration relative deformability of the facing elements and the reinforced soil mass and the backfill so that the correct magnitude and directions of wall friction forces would be used for the analysis of overall stability.

The work of Juran and Schlosser (1978) would provide the basis of limit analysis of reinforced earth structures.

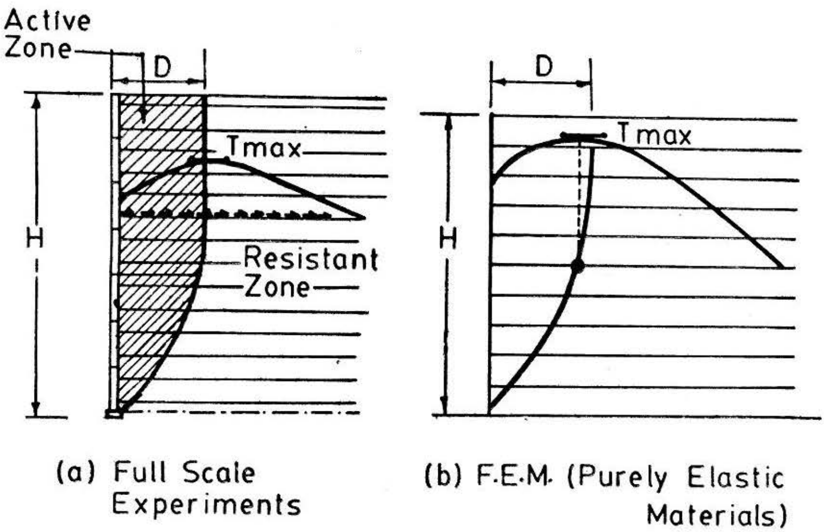
The limit analysis method of Juran and Schlosser assumes that the rotation of the active zone in the structure is large enough to generat the soil resistance to shearing along the potential sliding surface passing through the maximum tensile forces line.

The analysis, therefore, requires that the locus of the maximum tensile force in the reinforement is established and this locus would constitute a potential failure surface for the reinforcement and a sliding surface for the soil. Observations on actual structures and scaled models could be used with advantage to establish the geometry of this locus. Kinematical considerations are important since the locus must result in a system of compactible displacement and a velocity field consistent with the boundary conditions. Juran and Schlosser (1978) have suggested the use of a logarithmic spiral for this locus. The spiral passes through the facing toe and is vertical at the embankment free surface.

The tensile forces are determined considering overall equilibrium of the active zone situated between the facing and the locus of the maximum tensile forces. The soil reaction along the failure surface is determined by integration of Kotter's equation. It is assumed that the horizontal shear stresses on each horizontal plane, localised in the centre of any soil layer between two reinforcements are zero in the active zone, and thus the tensile forces may be determined by the horizontal equilibrium of each soil layer containing the considered reinforcement at its centre.

The theoretical results are in very good agreement with experimental observations on the locus of the maximum tensile forces (Figure 10) as well as with the distribution of the values of these tensile forces with depth.

Although the observations carried out by Juran and Schlosser (1978) are restricted to reinforced earth structures, where the reinforcement functions by shear resistance of adherence of the soil, it should be possible to adapt the above methods to reinforced structures where the reinforcement acts by confining the soil. Modelling, theoretical analysis and prototype trials are necessary to confirm the validity of Juran and Scholosser methods of analysis to soil reinforcements of different type from those used in the French practice. Anticipated behaviour of propose



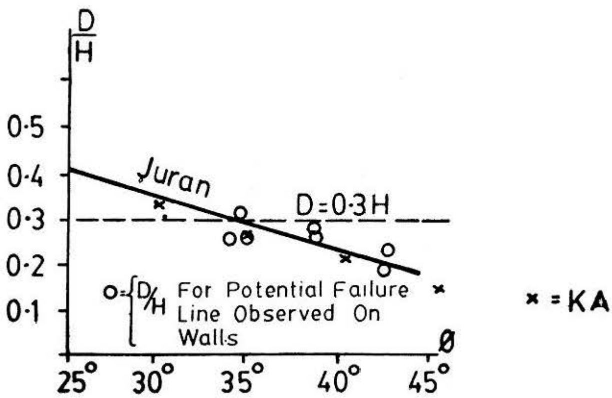


FIGURE 10 Tensile force distribution along reinforcements

system of soil reinforcement is described and a justification is furnished of the structural mechanism in Annexure 3.

Recommended Designs for Gravity Walls & Dams

The design is based on a coherent gravity structure hypothesis as shown in Figure 11. The reinforced soil mass is considered to behave like a gravity wall of identical external dimensions. Overall stability requirement with reference to the lateral forces due to earth pressure must first be satisfied. The choice of the base width is governed by this requirement of overall stability. In principle, a wall with sloping faces can be used. However, since the experience is limited to walls of constant width it is advisable to design walls of constant width until results of further research

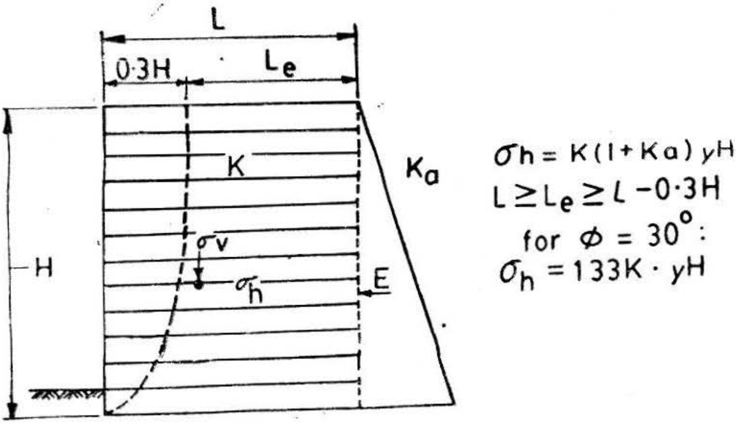


FIGURE 11 Coherent gravity structure Hypothesis for reinforced earth wall

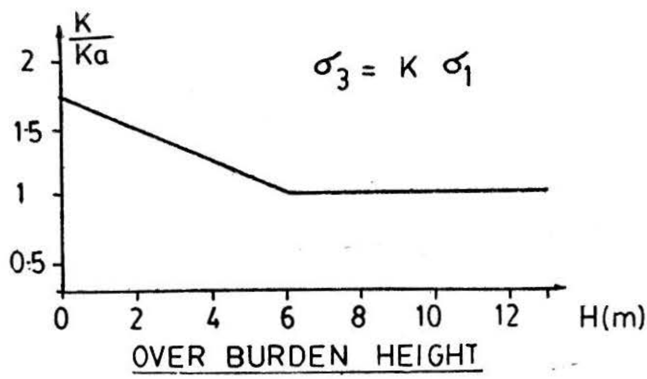


FIGURE 12 Distribution of tensile forces

on walls of variable width becames available. Results of sample calculation for a gravity wall are presented in Annexure 4. It would be seen that the base width for a wall for an active pressure coefficient of 0.33 would be 0.6 H for a factor of safety of 1 with regard to criterion of no tention at the base. The base width is increased to 0.8 H to provide a margin of safety with regard to tension at base and to make the overall dimension conform to reinforced earth walls successfully constructed in the past.

With regard to the reinforcement, the confining stress can be considered to be limited to $K_o \gamma H$. An element of uncertainty remains regarding distribution of tension forces in the proposed system of reinforcement. It may be presumed that the locus of maximum tension remains between a distance 0.2 to 0.4 H from the face of the wall. The tensil estrength of reinforcement may be assumed to be constant over the entire width. Comparative cost studies presented earlier in this chapter indicate that the proposed soil reinforcement system is economical as compared to conventional gravity walls of masonry even if reinforcement width is constant over the height of the wall. The basis of reinforcement provisions is explained in Annexure 1 and 2, It seems to be possible to modify the reinforement provision according to the probable distribution of tension forces. It should be reasonable to consider a triangular distribution on the basis of experience of reinforced earth walls (Figure 12). It is also possible to reduce the length of the reinforced zone towards the top of the wall while maintaining the base width at 0.8 H, without compromising the overall stability. Research has been initiated to make an evaluation of reinforcement system where the tensile resistance is varied according to the anticipated distribution of the tension force and the length of reinforcement is reduced in the upper part of the wall.

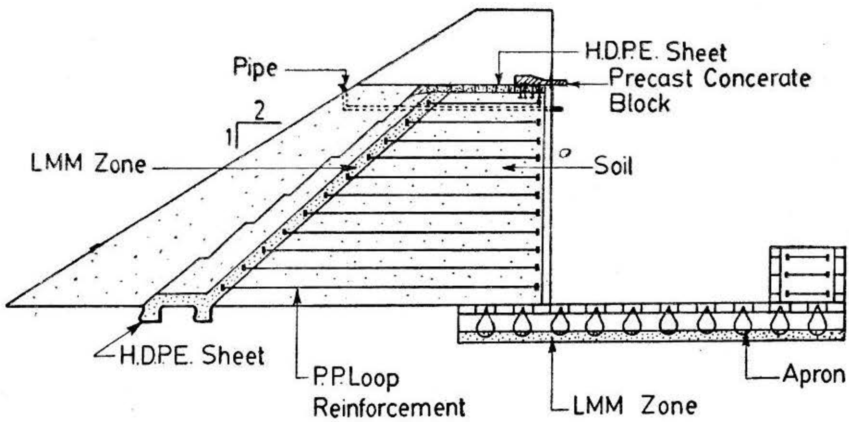


FIGURE 13 Reinforced soil over flow dam

This basis of dam design would be essentially similar to the design approach for gravity walls outlined above. Since experience of soil reinforcement system is limited to walls of constant width, it seems to be advisable to adopted a design for the earth dam wherein the reinforced section is of constant width and the upstream face has a sufficient slope to be stable without reinforcement. A small section of narrow width will be built of reinforced stabilised zone and this will be placed at the top of the section of reinforced soil. Design concepts for overflow dams based on above principle are illustrated in Figure 13.

Prospect for application of reinforced earth technique in India and identification of research needs

High cost of reinforcing materials has been a major constraint to introduction of reinfored earth techniques in India. Reinforced earth, compared to stone masonry construction, will continue to be expensive until materials e. g., galvanised iron strips and polymer fabrics, used abroad are replaced by low cost alternative materials.

Design have been developed which employ low cost facings and materials for reinforcement such as bamboo and commercially available strips of polymers. Experimental work essential for detailing of the proposed reinforcement systems has been completed. There is, however, an urgent need for field testing of the anchorages. When the designs of the anchorages are perfected after the required field trials, the proposed designs for retaining walls and low gravity dams can be taken up for prototype trials.

The basic research already carried out abroad is adequate for design of 'Type I' reinforcement. However, what is needed is the performance evaluation in the field, of instrumented prototypes. Developmental effort could best be concentrated on protective treatments and splicing methods for bamboo which has a very good potential for cost reduction and energy saving.

A new type of reinforcement system consisting of loops, spirals and rings is suggested and it is considered that this system has a very good potential for reducing the volume proportion of the reinforcement without compromising the performance.

Analytical studies and model experiments are needed to investigate the structural behaviour of the new reinforcement system and to establish design guidelines.

Annexure 5

Materials, Dimensions and Quantities for alternative designs of Reinforce Earthwall

The following three alternatives were examined

(1) Available reinforcement material i. e., Polypropylene strip loops.

Anchoring	Tiles of L.F.S.S. with fibre inclusion
Facing	Low Modulus Mortar & Concrete

This is the optimum design with energy saving components such as tile of estabilised soil with organic fibre for facing and anchorage.

(2) Available reinforcement material e. g., polypropylene strips and available material for facing and anchoring e. g., country tile, brick and rubble stone.

Dry brick wall with lean concrete bands is proposed for facing; alternatively, dry rubble facing may be used where available.

(3) RCC facing and galvanised iron strips as reinforcement (French practice).

Basic dimensions and Quantities for Alternatives I and II

Height of wall (M)	10M	5 M
Average width of wall (M)	8	4
ALTERNATIVE I		
Reinforcement ($1/m^3$)	0.54	0.216
Anchoring material Facing (Exterior face)	0.125 m^3/m^3	0.25 m^3/m^3

ALTERNATIVE II

1. Bricks for anchoring (Bricks thickness 5 cm L/M^3	50 Nos.	50 Nos.
2. Bricks for facing	0.125m^2/m^3	0.25m^2/m^3

Annexure

Basis of Estimation of Reinforced Earthwall

Basis of Estimation

	Rate	Reference
Alternative 1		
Polypropylene	Rs. 20/1it.	Market price
Compressed soil bricks	Rs. 25/M^3	Page IV
L.M.M.	Rs. 90/M^3	Page II
Facing	Rs. 5/M^3	Page III
Bricks size 20x10x10 cm.	Rs. 80/M^3	Market price
Alternative II		
Brick for facing	Rs. 120/M^3	Market price
Bricks facing (Average 8 cm thinkness including looping, bonds & mortar).	Rs. 12/M^2	Market price
Alternative III		
Galvanised steel strips	Rs. 10/kg.	Market price
R.C.C. (including reinforcing steel, form work, fixtures casting and placing)	Rs. 1500/M^3	Market price
Unit rate for 10 cm thick facing	Rs. 150/M^2	Market price

Unit Rate Analysis

Low Modulus Morter

Material	Qty. kg.	Unit rate Rs./kg.	Cost Qty. × Rate Rs.
Li *Lime*	80	0.7/kg.	56
Fly ash	80	0.2/kg.	16
Soil (Labour Expenses)	1m^3	.4/m^3	4
Net Material Cost/m^2			76
Labour for mixing, handling and placing			14
Net cost of placed material			Rs. 90/m^3

(Annexure 6—(Continued)

Clay tiles with Fibre Reinforcement

Size of Tils 20 x 10×2 cm. i.e., 2500 Nos./M^3

Material	Qty. kg.	Unit Rate Rs./kg.	Cost Qty.×Rate Rs.
Lime	90	0.7	63
Fly ash	90	0.2	18
Soil	1m^3	4.0/m^3	4
Fibre	22.5	2.0	45
Net Material Cost			Rs.130/m^3
Labour for mixing, handling & placing			Rs. 20/m^3
Net cost of finished material			Rs. 150/m^3
Net cost of finished material 2m thick			Rs. 3/m^2
Add for labour for placing			Rs. 2/m^2
Net cost of facing			Rs. 5/m^2

Compressed soil block

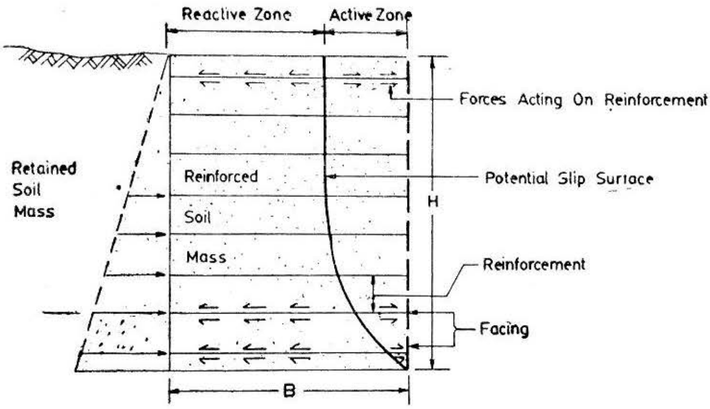
Material		Qty. kg.	Unit Rate Rs./kg.	Cost Qty.×Rate Rs.
Lime	) Loca) low grade	20	0.4	8
Surkhi	) materials	40	0.2	8
Soil		1 m^3	4.0/m^3	4
Raw material cost				20
Labour for making and other contingent expenses				5
Net cost of finished block				25/M^3

Annexure

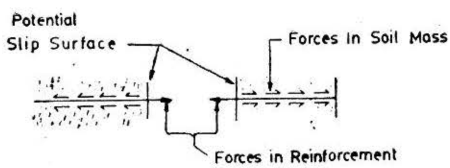
Structural Behaviour of Soil Reinforcement

The reinforced soil wall is presumed to behave as a coherent gravity structure. Reinforcements are closely spaced and are virtually inextensible thereby preventing lateral expansion of the soil. The facing would be flexible and it would not, therefore' restrain the settlement of the reinforced soil mass. No shear forces are, therefore, considered to act on the facing.

The reinforced soil may be considered to be made up of a number of elements with reinforcement placed along their centre line. At the interface of soil and reinforcement, sheer forces will be developed along the reinforcement. This force could be replaced by a single equivalent force acting on the facing along the centre line of the element. This is called as the 'confining force' and is represented by the forces F_1, $F_2 \ldots F_n$ in Figure A-2. The wall is considered to behave like a monolith composed of the



SECTION OF WALL



TYPICAL CASE OF FORCE EQUILIBRIUM IN A SOIL ELEMENT

FIGURE A-2 Section of wall and forces acting along reinforced soil mass and slip surface

quasi cohesive reinforced soil elements. The base width is so chosen that no tension develops on the base. When the reinforced structure is subjected to lateral pressure and gravity forces, an internal slip surface tends to develop. This divides the reinforced mass into an active and reactive zone. The horizontal force produced by the reinforcement is balanced by an equal and opposite earth pressure force or reaction acting on the sliding surface. Consequently, no horizontal shear exists in the active zone at the interface of the elements, i.e. midway between the reinforcements.

The forces acting on the individual elements and the active zone are shown in Figures A-3 and A-4. The shape of the slip surface may be a circle or a logarithmic spiral. The analysis is simplified by replacing the curved surface by two straight lines. (Figures A-4_b)

The reactive zone can be simplified into a body of simple geometry subjected to external forces consisting of the earth pressures from the slip surfaces and the forces of the reinforcements and the gravity and earth pressure of the backfill. The earth pressure can be replaced by a set of

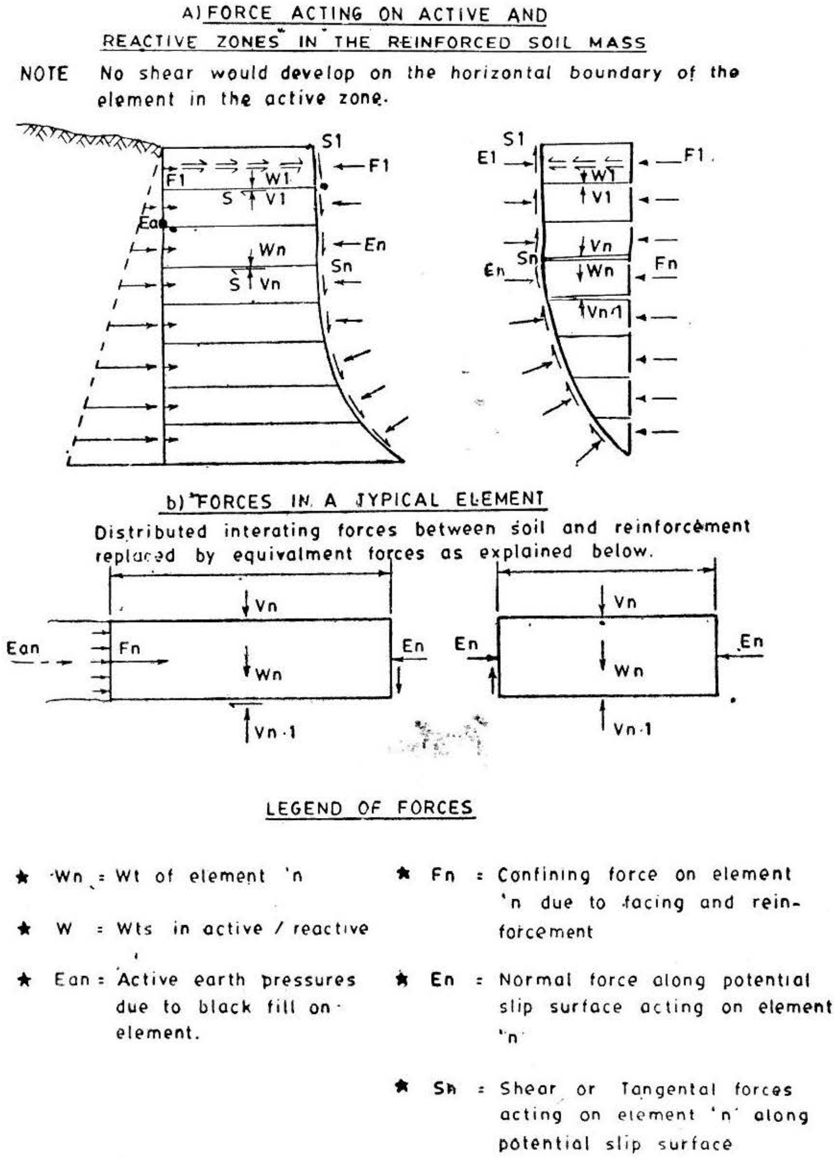
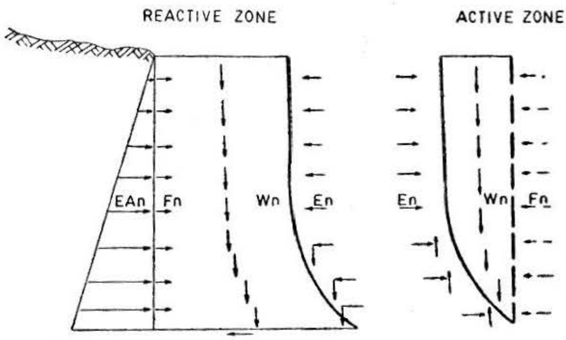


FIGURE A-3

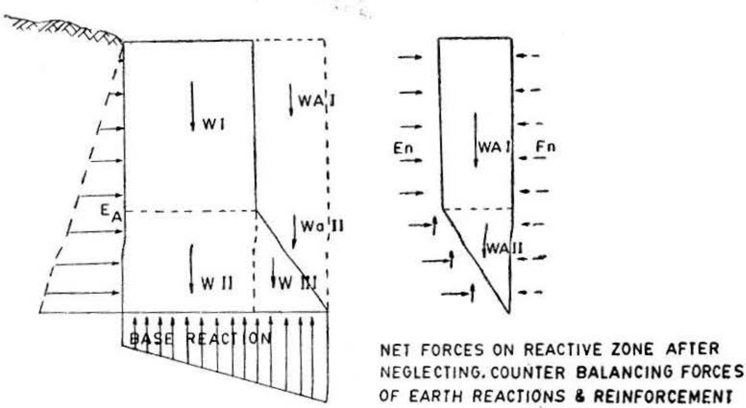
horizontal forces which are equal and opposite to the confining forces F_1, F_2 etc. and vertical forces equal to the gravity forces corresponding to the weight of the various elements in the active zone. The forces of reinforcement are equal to the confining forces F_1, F_2 etc. and, therefore, balance the horizontal component of the earth pressure reaction on the slip surface.

The reactive zone also functions as a coherent mass retaining the soil backfill and subjected to the lateral earth pressure.

Shear stresses in the active zone are only due to the reinforcement, while in the reactive zone, shear due to lateral forces (i.e., the active pressure of the backfill) must also be considered.



a) FORCES ACTING ON SOIL ELEMENTS IN ACTIVE & REACTIVE ZONES



b) SIMPLIFIED MODEL OF FORCES

FIGURE A-4

Since K is usually of the order of 0.33, the friction factor would be about 0.2. Most soils, except plastic clays, if drained properly can have the required shear resistance.

Annexure

$^*\frac{B}{H} = 0.8,\ K_A = 0.33$, Max $\sigma_3 = 8\text{T/m}^2$

*Allowable stress = 1000 kg/cm²

*Wall friction Ignored.

Bending moments $= \frac{1}{6} K_A \gamma H^3$

Moment of Resistance $= \frac{1}{6} B^2$

Bending stress $= \frac{K_A \gamma H^3}{B^2} = \frac{\gamma H}{1.92}$

Direct stress $= \gamma H = 16$ T/m²

Thus, bending stress $= \frac{16}{1.92} = +8.33$ T/m²

Friction Factor $= \frac{\text{Horizontal Forces}}{\text{Weight}} = \frac{0.5 K_A \gamma H^2}{B\gamma H} = \frac{K_A}{2 \times 0.8}$
$= 0.625 K_A$

Reinforcement arranged as per σ_3 distribution.

The reinforcement requirements per unit volume after allowing for + 5 per cent variation are shown in (*b*).

Reinforcement volume proportion $- \frac{8+1.2}{1000} = 0.096$

Total Reinf. =0.719 litres/m³ of wall volume.

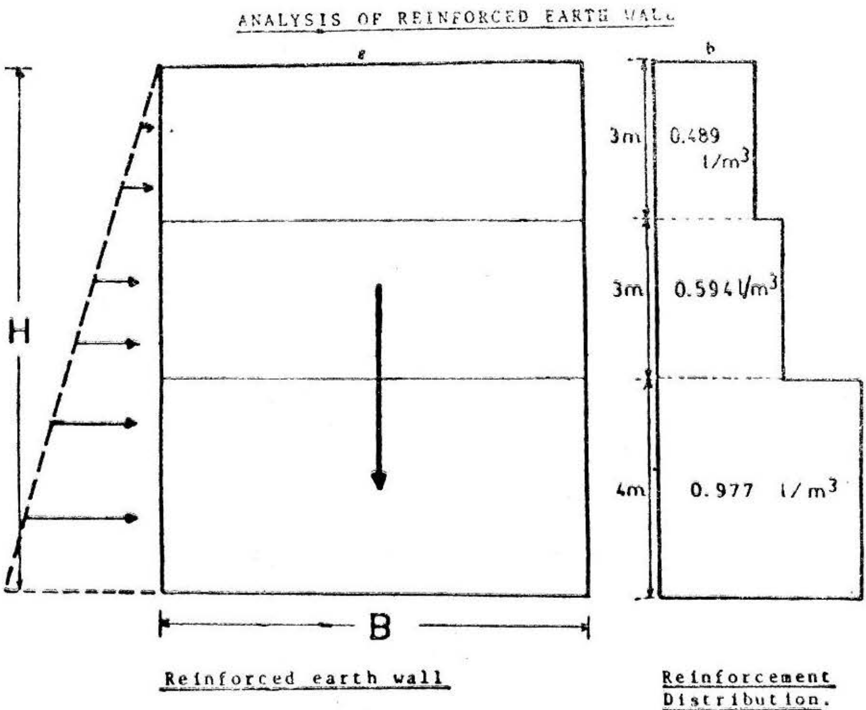


FIGURE A-5

Stabilised soil with Fibre Inclusions

An alternative approach to soil stabilisation

It is several years since researchers first established that lime is an ideal material for soil stabilisation under Indian conditions. Extensive laboratory testing had confirmed the potentiality of lime stablisation for a variety of soil types ranging from silts to highly plastic clays and expansive soils, yet findings of laboratory research have not been applied so far on any significant scale.

The main constraints to practical application of stabilisation techniques in India has been the cost and availability of equipment for processing, mixing and compaction of soil in the field. In practice, without mechanisation. it is difficult to obtain the desired characteristics with regard to density and moisture content and to ensure uniformity of mixing in order to attain the required quality standards. Thus, the energy saving achieved by the use of lime would be offset by the energy consumed by mechanical equipment required for mixing, handling and compaction.

I, therefore, thought of exploring alternative approaches viz., use of soil with lime puzzolana admixture instead of quick lime or slaked lime. Attention was concentrated on the improvement of internal friction properties along with a small improvement in cohesion. The possibility of fibre inclusion was also examined in conjunction with lime fly ash stabilisation. At the outset, I found that a wet mixing process could be used where lime puzzolana could be added in the form of a slurry. Further testing revealed that improved performance could be achieved by use of premoulded products with a lower moisture content. With warm water curing, strength development could be accelerated so that the products could be handled within a few days of manufacture. Preliminary experiments confirmed that accelerated curing could be quite effective with warm water at 70 to 80 C. Solar heating would be feasible for curing since available designs of solar heating equipment could be adapted to field curing applications.

An advantage of pre-moulding or pre-casting is that better quality control could be exercised. By the use of suitable moulding machines, the problems of availability of field compaction would be overcome and compaction energy input can be reduced. Lime fly ash stabilised soil with fibre inclusion would thus be superior to field mixed stabilised soil.

I would like to clarify that stabilisation of highly cohesive soils and black cotton soil is not considered at the present stage since the major applications are intended to replace energy intensive materials such as concrete and pricks which involve heavy energy consumption in manufacture, processing and transportation. Stabilished soil premoulded products with natural fibre inclusions are also proposed to be used for soil reinforcement systems as facing and anchorage elements. The development of stabilised soil with fibre inclusion was, therefore, considered as an essential preliminary to evolution of low cost soil reinforcing systems.

Fibre Inclusion

Fibre inclusion constitutes an effective means of imparting cohesion to premoulded and compacted products. When fibres from renewable sources are used, energy saving would be very large relative to other methods of soil stabilisation which rely on energy intensive materials such as cement and petrochemical products.

In order to optimise the system of fibre reinforcement it is necessary to understand the role of fibrer inclusions and therefore, a resume of the result of previous studies on fibre inclusion is presented in the following paragraphs. I have relied mainly on the general report of Mitchell and Schlosser (1979).

Improvement of soil properties by inclusion of fibres has been investigated in India as well as abroad and it has been established that strength and deformation resistance of soil can be improved by fibre inclusions. However, previous investigators do not seem to have considered practical problems of manufacture and field placement. The economics of the fibre reinforced soil as a construction material does not seem to have been considered with regard to the choice of the reinforcing material and the energy consumption in construction operations as well as manufacture of materials.

From the point of view of energy consumption, it is obvious that natural fibres should be preferred to asbestos, glass or polymers. Laboratory research was, therefore, directed mainly to the use of natural fibre Fibre inclusion can be divided into two categories :

—Random oriented fibres mixed with the soil.

—Oriented fibres e.g: layers of woven fabric placed within the soil.

The behaviour of the fibre reinforced system very much depends on the density of the fibre reinforced material, as can be inferred from the experience of fibre reinforced cement products where random inclusions of natural fibre has been considered for boards, roofing sheet and pipes. Compaction has been found to be essential; otherwise strength improvement may be small or insignificant.

In this context, it should be noted that organic fibres natural or artificial have been found to be very effective for improving impact resistance of concrete even though they were of very liwited benefit for improving the flexural resistance of reinforced members subjected to normal static or repetitive loading.

Test on stabilised soil with fibre inclusion

The posibility that fibre inclusions might improve the strength and deformation resistance of soils has been considered previously by Hausmann (1976). Hausmann studied in the triaxial apparatus the effect of the diameter of inclusions on the failure by lack of adherence. He showed that the appatent friction angle increases with the dimension of the inclusion (Figure 14).

Both drained and undrained trixial tests were done on mixtures of kaolinite and pulp (almost pure cellulose) fibres by Andersland and Khattak. The fibres had a weighted average length of 1.6 mm and a typical diameter o: 0.02 mm. Fibre contents of 16 and 40 per cent by dry weight were used, and test specimen were consolidated from a slurry.

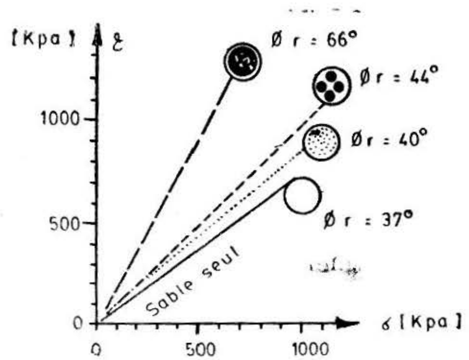


FIGURE 14 Variations of with the dimension of the inclusion (Hausmann, 1976)

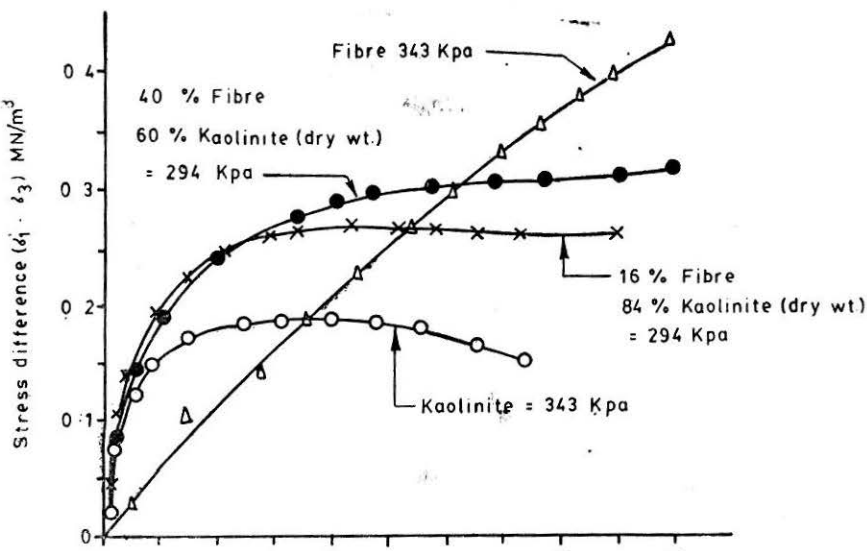


FIGURE 15 Stress-strain curves for undrained conditions and samples with different kaolinite/fiber comcinations

The addition of fibre significantly increased the strength of Kaolinite for undrained loading conditions, as may be seen in Figure 15, which illustrates also the influence of the fibres on the stress-strain behaviour. The reinforcing effect of the fibre is further illustrated by an increase in effective stress friction angle from 20° for pure Kaolinite to 39° for 100 per cent fibre samples. It should be noted that the fibre content for random oriented fibres is limited by practical difficulty of mixing since the fibres tend to accumulate and ball up so that a uniform distribution is difficult when the fibre content exceeds 5 per cent by volume, Results of laboratory studies on improvement of soil properties by fibre inclusions therefore lose their relevance in practical applications for random oriented fibres when the fibre content exceeds 5 per cent. Test results summarised by Mitchell and Schlosser are of practical value only if fibre can be used in the form of of mats, nets, or woven fabrics placed within the soil layers.

The conclusion that emerges from the above review is that fibre inclusion would be very useful and effective in improving the properties of premoulded products of stabilised soils. The improved characteristics will be as follows:

— higher impact resistance

— ability to withstand handling stresses

— increased shear strength especially with compaction and compression at sufficiently high pressures.

There is, however, a very interesting prospect of making stabilised soil blocks with layers of woven fabric placed at regular intervals. (Brams 1977, 1978). Oriented fibres would further increase the flexural strength if the reinforcement is suitably disposed.

Proposed uses of stabilised soil with fibre inclusions

There is good potential for use of lime-fly-ash-stabilised soil with fibre inclusion made in the form of premoulded, compressed pre-cured blocks, slabs, tiles and bricks. These would replace bricks and concrete blocks. Energy consumption in the pre-cast products would be only 5 to 10 per cent of brick or concrete blocks.

Where haulage and transport of flyash is uneconomical, it may be possible to use local reactive material such as Surkhi when suitable clays are available. A blend of Surkhi of law reactivity with Rice husk ash may also be considered if reactive raw material is not available for Surkhi manufacture. Grinding of lime and Surkhi is of vital importance for attaining the required strength of the lime puzzolana stabilised soil and ball mills should be used for grinding.

A range of soil types consisting of sands, silts and lean clays have been found to be suitable for fabrication of lime flyash soil products with fibre inclusion. This opens up a wide field of application since the cost of quarrying aggregate processing and transport would be virtually eliminated. For rural development transportation distance would be in the range of haulage by bullock carts. For some of the major irrigation works and road construction, soil available within haulage range of bullock cart could possibly be used. Potential applications would consist of :

— Blocks and bricks for walls, arches, columns and piers.

— Lining of small and medium size channels,

— Components for construction in reinforced soil consisting mainly of facing and anchorage elements.

— Pavement for law traffic volume roads.

There is an attractive prospect for use of blocks of this material in conjunction with reinforcement of strips of bamboo and other organic fibre for soil, storage structures, domes and tanks. Some of the above applications in the premoulded products could be used in conjunction with reinforced stabilised soil placed in-situ.

Research need for soil stabilisation

With regard to soil stabilisation, the research needs fall in thsee categories : (*i*) Improvement of mixing methods; (*ii*) Choice of puzzolonic material with a view to minimısing energy consumption in manufacture, processing and transportation; and (*iii*) Investigating alternative admixtures having potential for cost reduction and energy saving.

(*i*) *Mixing* : Essentially, this implies modifications of concrete mixers to increase the output and reduce energy consumption. Non-

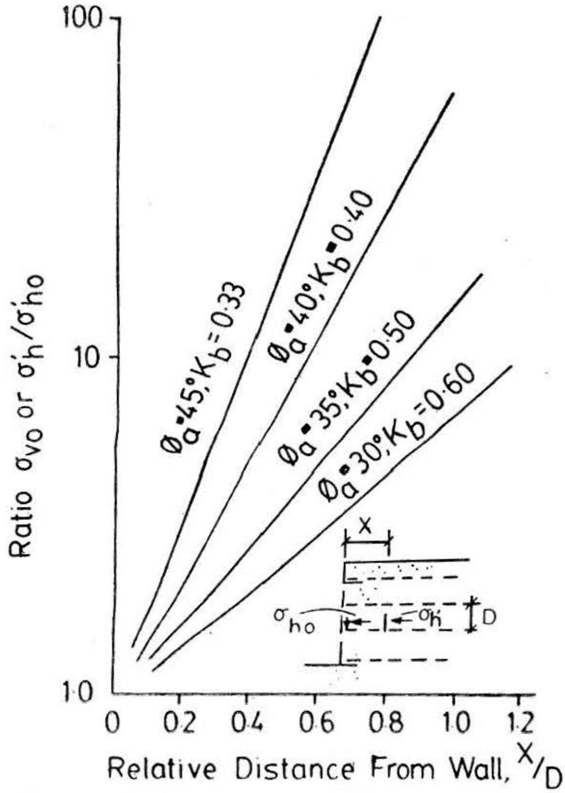


FIGURE 16 Increase of σ_v/σ_{vo} and σ_n/σ_{no} with increasing distance x/d from the retaining structure (Broms 1978)

tilting mixers seem to be the most economic. Field experiments should be directed to establish the minimum period of mixing for various mix proportions and the influence of variations in content of the lime fly ash on the angle of internal friction of the lime fly ash stabilised soil.

(*ii*) *Puzzolonic Materials* : The laboratory studies be directed to the nature of the lime soil fly ash reaction since the puzzolonic activity of natural clays can be availed of to minimise the lime fly ash content, and it has been found that the puzzolonic activity has no relation to the engineering properties. (Carvalho 1981).
Utilisation of rice husk ash, pretreatment of natural puzzolonic clays by heating at law temperature, and mixing of fly ash with finely pulverised Surkhi of moderately reactive clays are energy saving possibilities, which should be investigated. The economics of these materials would vary according to the local situations ; and, therefore, regional studies are essential.

(*iii*) *Other stabilising admixtures* : Among the recently developed admixtures, use of iron oxide merits serious consideration because of its potential for energy saving. The process involves heating a fine graincd soil to a temperature high enough to destroy its water sensitivity and adding to it finely divided iron oxide and adding sodium silicate solution. (Ingles and Lim 1980).

Reinforced Stabilised Soil

Approach

After having estabilished the feasibility of low cost reinforcement systems with strips of bamboo or polymers, an evaluation of the economics of soil reinforcements was taken up. It was evident from a preliminary analysis that the volume proportion of reinforcement and the cost of reinforcement component per cubic metre of a gravity wall increase almost directly in proportion to height. Thus the situation with regard to relative cost of masonry and reinforced soil walls changes radically as the wall height exceeds 15 m and the reinforced soil system tends to become uneconomical at about 20 m height.

Another factor restricting the use of soil reinforcement is the need for soils with adequate internal friction. Cohesive soils are subject to volume change and are likely to develop residual stresses at the end of each cycle of moisture variation. In practice, there could be several cycles of moisture change depending on the number or wet spells and seasonal variations of ground water level or reservoir level.

It was, therefore, considered necessary to investigate alternative soil reinforcement system which are not subject to the above inadequacies. Bearing in mind the overall objective of cost reduction and energy, saving reinforced stabilised soil seemed to be an obvious choice. Attention was concentrated on low cost admixtures such as lime and fly ash and a wet consistency was preferred which corresponds to the consistency of mud mortar commonly used for low cost constructions.

Soil types investigated included cohesive souls of low plasticity mixed with locally available low grade aggregates normally unsuitable for cement concrete and masonry construction. Stabilisation of highly plastic soils was not considered since such soils would require a high Lime content for achievement of desired permeability and internal friction characteristics.

Laboratory Studies

Investigations were carried out by conducting triaxial tests on cylindrical specimens reinforced by providing a winding of reinforcing wire.

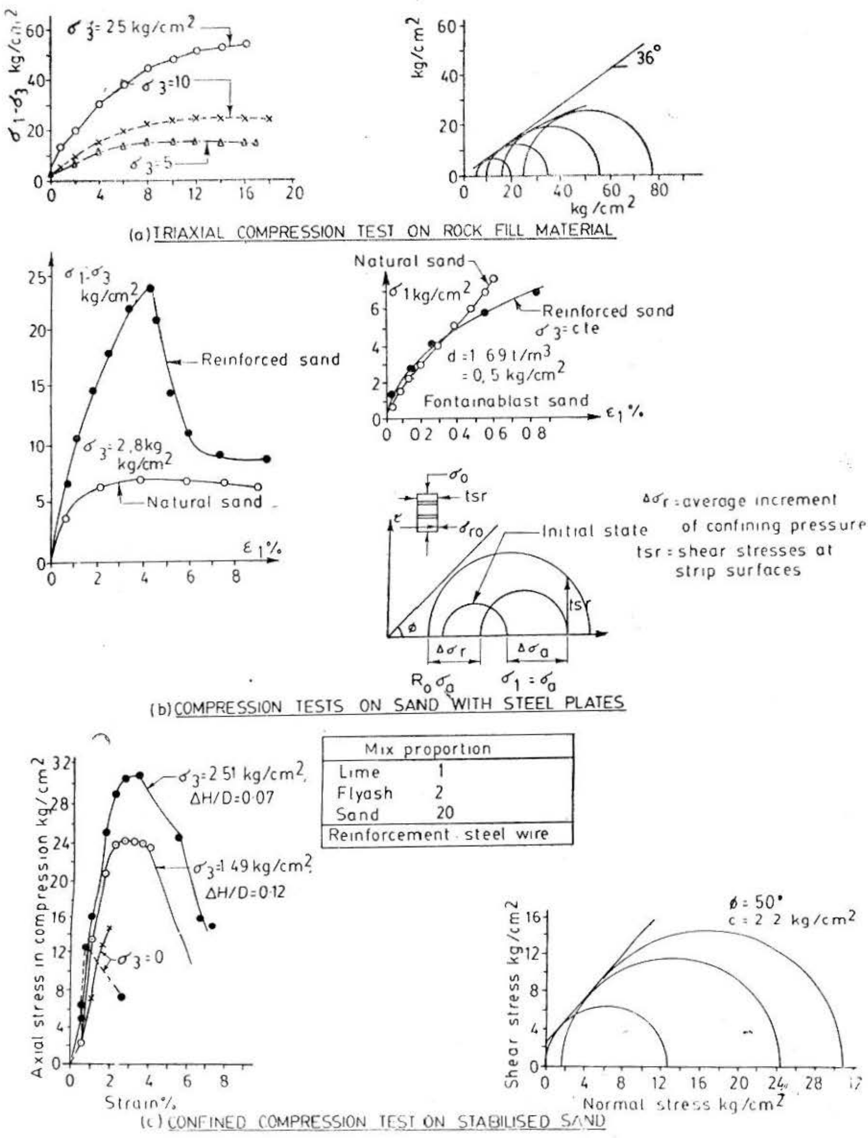


FIGURE 17 Laboratory tests on reinforced stabilised soils

The mode of failure of the reinforced stabilised soil specimens was examined and it was found that failure generally occurred due to breaking of the reinforcement. The tensile strength of the reinforcement is believed to be fully mobilised and, therefore, it is implicitly assumed that the yield stress in the test represents the situation where lateral confining stress would be equal to ultimate bursting pressure of a confined cylinder. The benefit of the reinforcement is presumed to be available uniformly over the entire height of the cylinder and assuming this, the confining capacity of the reinforcement will be T/r.p. where T is the ultimate tensile strength of the wire, r is the radius of the specimen and p is the pitch of the reinforcing spiral or the spacing of the rings.

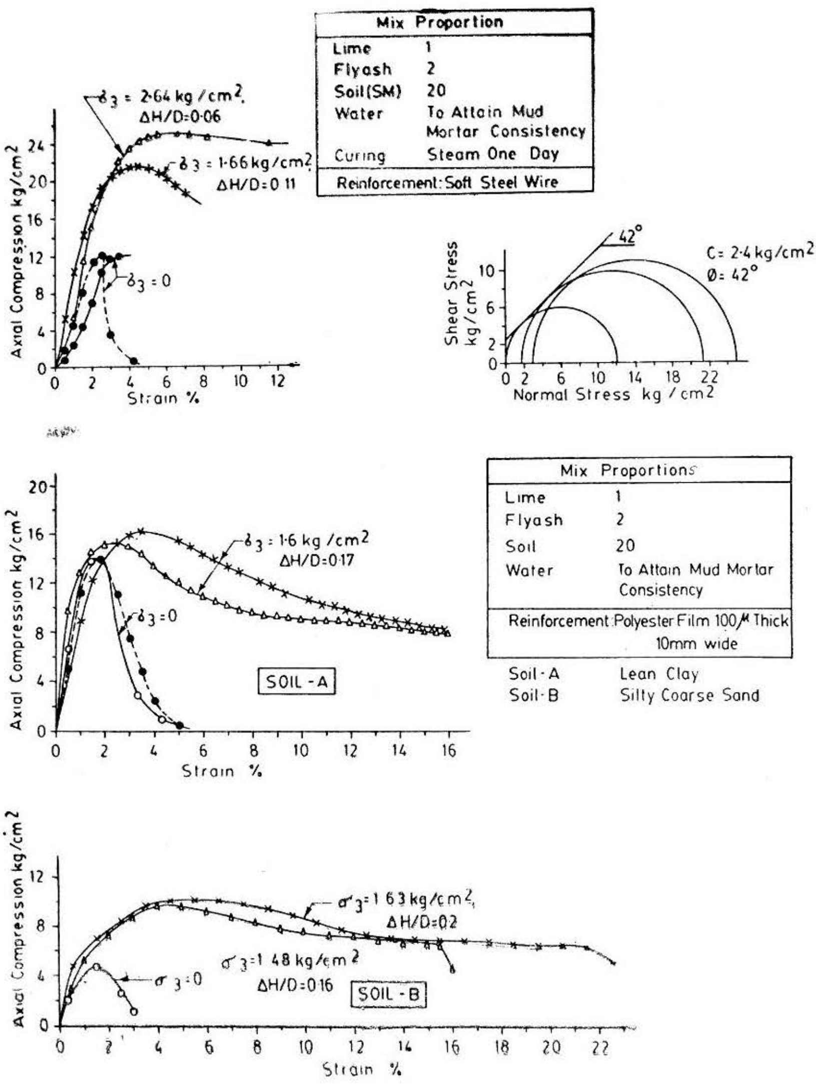


FIGURE 18 Laboratory tests on reinforced stabilised soils

In Figure 17, comparison is made of the results of tests on standard sand lime fly ash admixture with triaxial tests on rock fill reported by Marsal (1967) and test on reinforced sand reported by Juran et al (1978). There is a remarkable similarity in the behaviour of these materials. viz. sand subject to a lateral pressure corresponding to the coefficient of earth presure at rest, or the condition of low lateral yield, and the test results for rock fill or the reinforced sand or the reinforced lime fly ash stabilised soil. The type of reinforcement does not seem to be of any consequence as would be seem from the comparison of the behaviour of sand specimen reinforced by a series of parallel discs and stabilised soil specimen reinforced by a winding of steel wire.

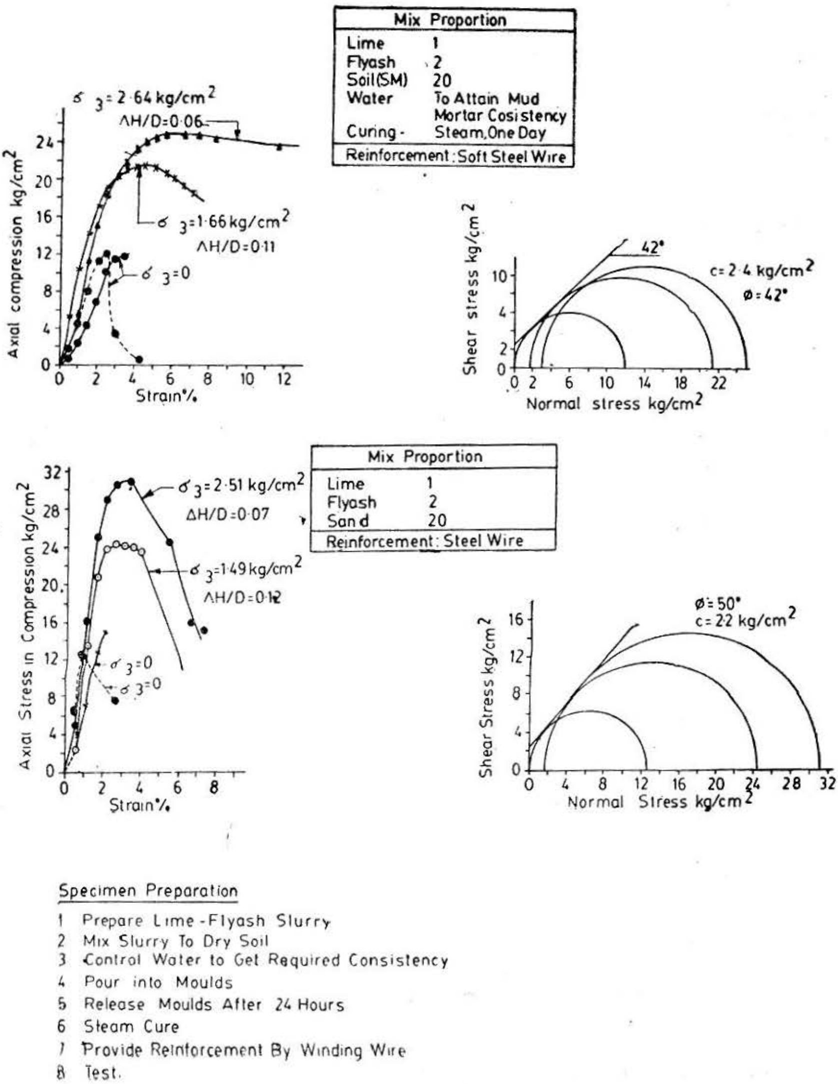


FIGURE 19 Laboratory tests on reinforced stabilised soils

The reinforcement makes a major contribution to axial strength and imparts ductility, so that the reinforced material can, before rupture occurs absorb several times the strain energy at working stress.

In the second series of tests a comparison is made of the behaviour of stabilished soil reinforced by steel wire and stabilised soil with reinforcing rings of polyster film. The improvement in ductility by the use of polyster reinforcement is quite evident as may be seen in Figure 18.

In the third series of test, a comparison is made of the behaviour of reinforced lime-ply ash stabilished soil and reinforced sand with lime-fly ash. (Figure 19). It would be seen from the test results that a very significant improvement in the internal friction and cohesion of the soil is achieved with the addition of lime and fly ash in a small proportion. A noteworthy feature of the test results is that the Mohr Coulomb envelope maintains a high angle of internal friction even for stresses as high as 24 kg. per cm^2.

Characteristics of reinforced stabilised soil

From the laboratory investigation, it was evident that reinforced stabilised soil is a low cost material with remarkable properties. The following characteristics make this material eminently suitable for construction of gravity structure as well as several other applications.

(*i*) Value of angle of internal friction ϕ is 40 or better and cohesion C values of about $2kg/cm^2$ can be achieved with lime contents as low as 60 kg/m^3 for a variety of soil types ranging from lean clays, silts, silty sands, dirty sands and gravel, murum, inferior aggregates such as laterite. The performance of the lime fly ash stablished soil does not seem to be sensitive to variations in the mix proportions. Properties of the soil with regard to internal friction appear to be quite consistent.

(*ii*) The values of the compressibility and angle of internal friction can be controlled and the range of their variation can be narrowed down so that field control would be easy; yet a wide range of locally available materials can be used.

(*iii*) Lime fly ash stabilised soil has ϕ exceeding 40° for stresses as high as 24 kg/cm^2. Rock fill on the other hand suffers from disadvantage of reduction of ϕ values at high stress levels (Figure 20).

(*iv*) Another feature of the lime fly ash stabilised soil is that a very small extent of reinforcement is required to improve the axial compressive strength of the material and a confining capacity of only about 10 per cent of the maximum axial compressive stress is sufficient.

(*v*) The permeability has been found to be of the order of 10^{-4} cm/sec. and a possible lower limit would be 10^{-5} cm/sec. The material would be impervious enough to control seepage and prevent build up of pore pressure if used in conjunction with suitable filters and drains. A permeability of the order of 10^{-4} cm/sec. and a compressibility of the order of 1000 kg/cm² implies a coefficient of consolidation exceeding 1000 m²/year. Build up of construction pore pressures is, therefore, ruled out.

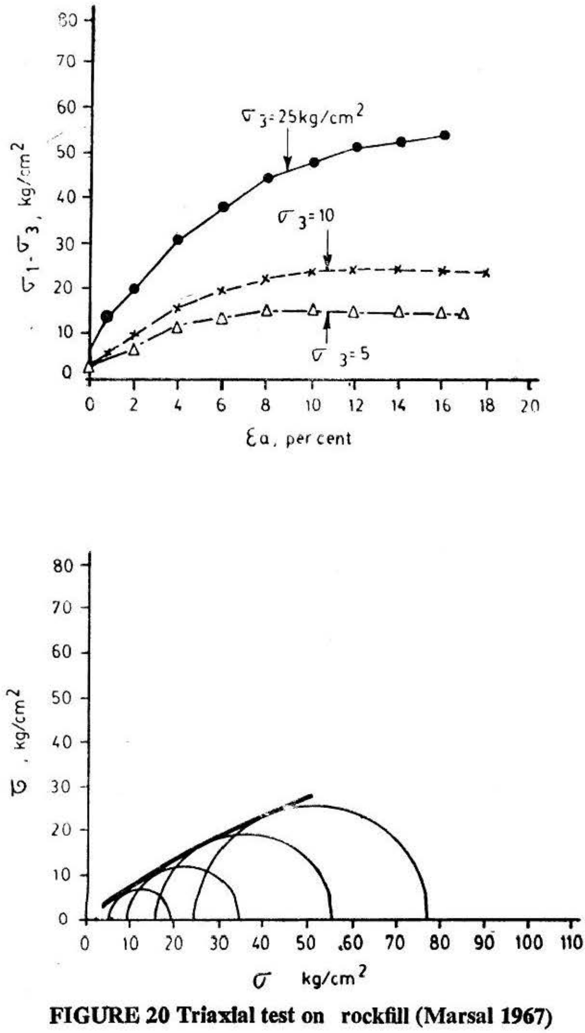


FIGURE 20 Triaxial test on rockfill (Marsal 1967)

(*vi*) The material would be very easy to mix and place since a wet consistency is quite suitable and vibration is not necessary due to the fluidity of the mix.

Energy saving by use of tensile reinforcement

It can be derived from results of laboratory test that about a litre of reinforcing material having an ultimate tensile strength of 1,000 kg/cm² can improve the compressive strength of the stabilised soil by about 5 kg/cm². Cementatious materials are relatively inefficient, since about 150 kg.

of cement must be added to a cubic meter of concrete to increase the strength from 200 to 300 kg/cm^2 i. e., about 1.5 kg/m^3 to added 1 kg/cm^2 to the strength. If natural fibre can be used, the energy saving would be enormous since apart from energy consumed in manufacturing there would also be the saving in the transportation. However, such a direct comparison cannot be used as a basis of comparative evaluation of alternatives until preservative treatments are developed to ensure a reasonable working life for construction with natural fibre reinforcement. The comparion is, however, indicative of the potentially for the natural fibres as a construc-

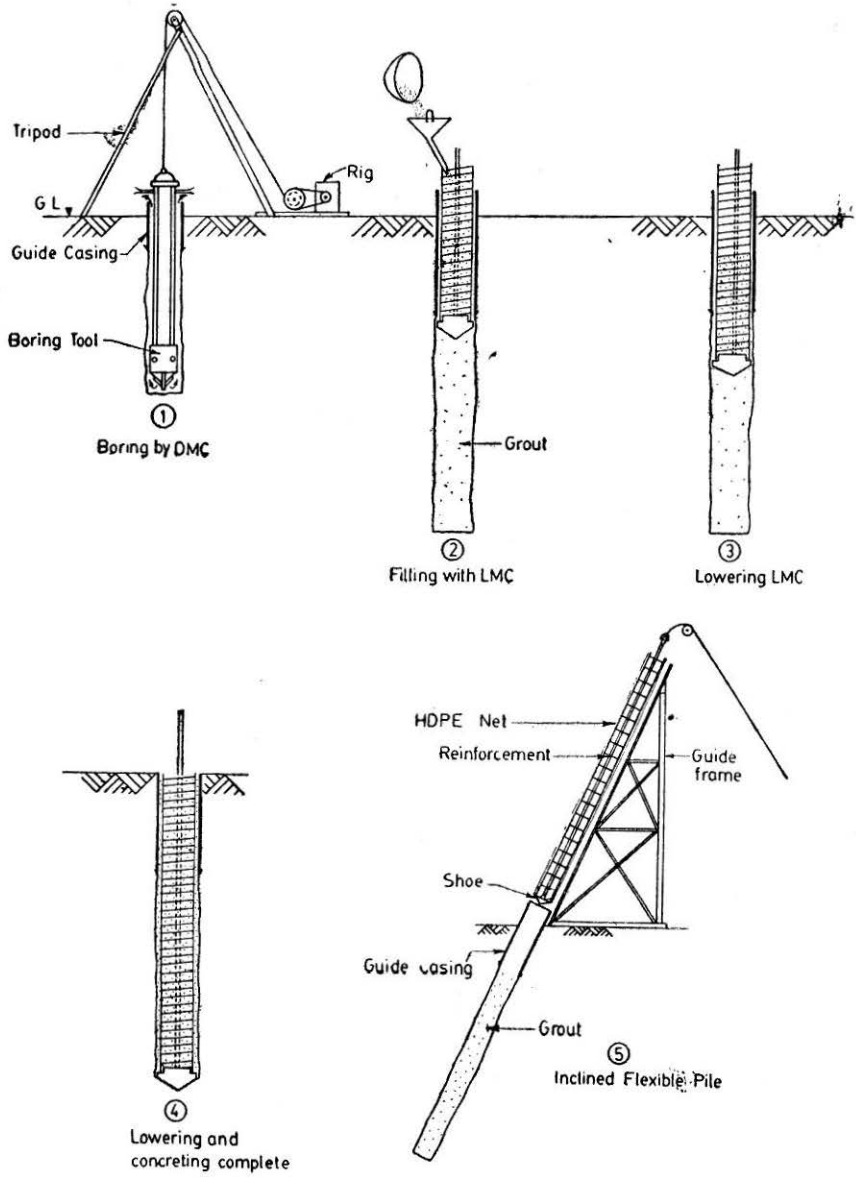


FIGURE 21 Low modulus concrete piles

tion material and the need for intensifying research efforts in the development of techniques for using natural fibre reinforcement.

Use of reinforced stabilised soil for diaphragm walls and piles

By virtue of their flexibility, polymer reinforced elements of cement fly ash concrete or self setting slurries have a good potential for application where the rigidity of reinforced concrete of normal grades is a disadvantage.

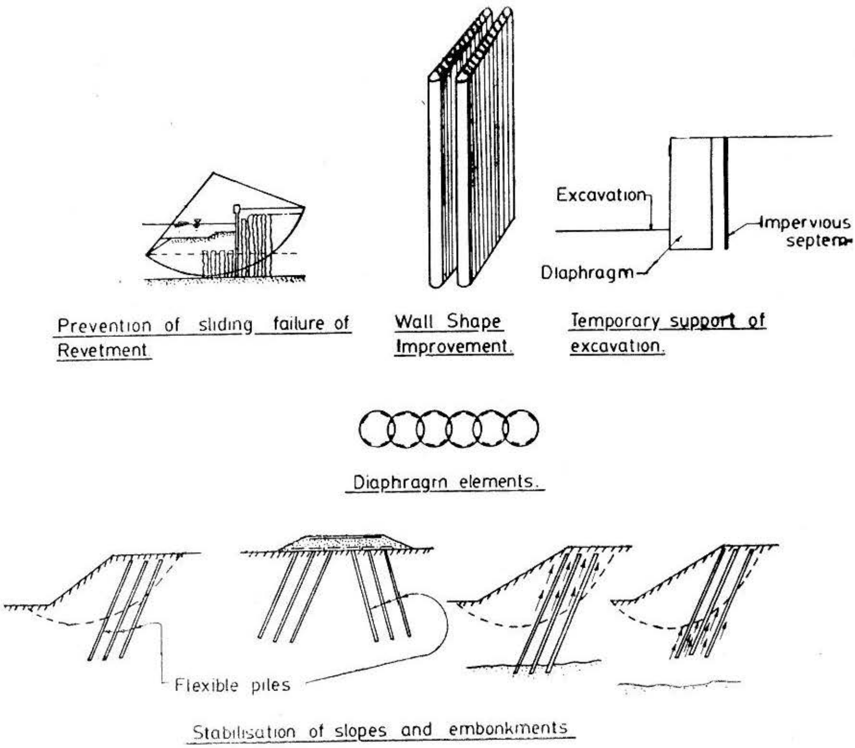


FIGURE 22 Applications of reinforced stabilised soil piles and elements

For example, a pile of reinforced low modulus lime fly ash concrete would be a very effective means of reinforcing soil. In soil reinforcing application, the load carried by the pile will be limited by skin friction transfer. Therefore, strength of normal concrete cannot be fully utilised. An optimum ratio of skin friction to axial capacity can be achieved probably at an allowable stress of 10-15 kg/cm². A further advantage of this system is that a vertical or inclined pile can be installed. Methods installation are illustrated in Figure 21.

Elements of reinforced lime fly ash concrete

Low cost diaphragm techniques based on use of jetting tools such as the methods developed by CBRI can be used to instal elements for soil improvements in loose sand and soft cohesive strata. These elements would be of rectangular shape and the load transfer would be achieved mainly by skin friction.

Potential applications are illustrated in Figure 22. Polymer reinforced diaphragm elements can be very effective in resisting lateral forces for temporary support of excavation, or relieving the loads on sheet pile walls, and stabilisation of slopes or forming bases of coffer dams and breakwaters.

Optimum Gravity dam

Background

Construction of dams involves use of energy intensive materials such as cement and heavy equipment for earthmoving, concrete mixing, batching and handling. Realising this, I embarked on a search for low cost energy saving alternatives with the hope that if a feasible alternative can be found, its contribution would be of great significance in overcoming constraints to hydropower and water resource development. At the outset, it was evident that the present design practice for gravity dams suffers from a major limitation that the allowable stress in masonry or concrete is related to the unconfined compressive strength. Further the compressive strength of stone material is fully utilised only in dams of about 50 M. height with conventional factors of safety. In low dams, the strength available exceeds the structural requirement by a large margin, yet a minimum cement is stipulated mainly from the point of view of seepage control and prevention of leaching.

Search for an optimum section

The first step in the search for an optimum section and material was to find a way of removing the anamoly with regard to the strength criteria for concrete material and earth viz., the stability analysis of earth structures is based on the shear resistance while the unconfined compressive strength governs the design of concrete or masonry structures. Designers often seem to lose sight of the fact that soil is a plastic material which continues to deform after reaching the yield stress. Consequently while higher factors of safety are essential for brittle materials, factors of safety as low as 1.5 are acceptable if the strength parameters are chosen judiciously so that the design value corresponds to the minimum shear strength that can be obtained under field conditions.

From the above discussions, it is evldent that the optimum material for a dam would be a ductile material with high shear resistance. I found that lime fly ash stabilised soil has several advantages from this point of view.

—Its internal friction characteristics are remarkably consistent and a minimum ϕ value ranging from 40° to 45° can be realised for a wide range of soils.

—The ϕ value for reinforced stabilised soil does not decrease even for stress levels of 25 kg/cm^2 or higher. In this respect the lime fly ash stabilised soil is superior to rock fill since the ϕ value for rock fill decrease with increasing stress level due to crushing of internal granular contracts and it could drop to about 35° for stress levels of 30 kg/cm^2.

—Reinforced lime fly ash stabilised soil has high ductility and compressive stresses as high as 20 kg/cm² can be sustained even with a very low content of lime and fly ash and a very small volume percentage polymer reinforcement.

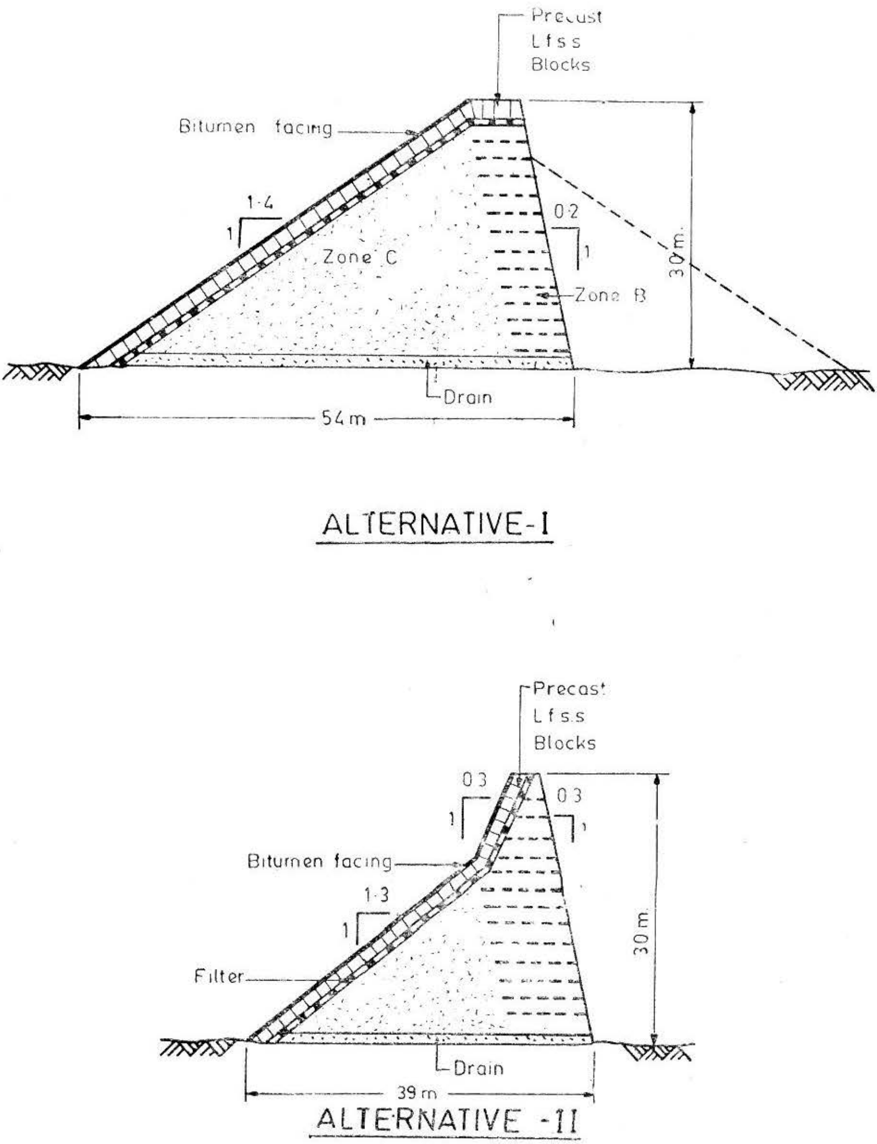


FIGURE 23 Sections of reinforced stabilised soil dams

If advantage can be taken of confining stress by selecting a suitable geometry and by providing reinforcement, use of lime fly ash stabilised soil can very well be extended to dams where the compressive stress exceeds the unconfined compressive strength of the material by a large margin. It may be noted here that the confining capacity of the reinforcement need only be 10 per cent of the maximum compressive stress. Therefore, a volume proportion of only about 1 litre of reinforcement per cubic metre of the stabilised soil mass is sufficient for a working stress of 10 kg/cm^2 with a factor of safety of 2 of the reinforcement (i. e., with an allowable stress of 1000 kg/cm^2 for poly-propylene strips).

I undertook a comparative evaluation of alternative designs with reinforced earth and with reinforced stabilised soil. As compared to the reinforced stabilised soil, reinforced earth technique requires a larger volume proportion of reinforcement since the level of the confining stress required is as high as 40 to 50 per cent of the vertical stress. In reinforced lime fly ash stabilised soil, on the other hand, the confining stress level need only be about 10 per cent of the vertical stress. Hence use of reinforcement would be limited to only a part of the section of the dam by choice of an appropriate geometry. Typical sections are shown in Figure 23 and 24. Quantities and costs of soil and reinforcing material are considered.

Limitations of the two systems from engineering point of view and energy consumption are compared below :

Limitations	Energy Considerations
Reinforced Earth	
* Uncertainty regarding long term durability and performance of reinforcing material. * Lack of performance experience of reinforced earth dams. * Experience limited to walls with strips of constant width. * Lack of evidence regarding walls with variable length of reinforcement over the height.	* Extent of saving decreases with increasing height of wall and may be marginal when wall height exceeds 30 meters. * Small saving if energy intensive material is used for reinforcement.
Reinforced Stabilised soil	
* No previous experience of system and material performance e.g. durability of reinforcement and leaching of lime and fly ash.	* For wall height exceeding 15 or 20 m significant energy saving reinforcement if used. * Energy input in lime-fly ash and reinforcement is small as compared to concrete and masonary dams.

While it must be acknowledged that there is no experience of construction of reinforced lime fly ash stabilised soil, it should be possible to extrapolate the experience of several thousand structures up to 30 m height of reinforced clean sand and gravel. The stabilised soil would have low compressibility and a very consistent behaviour regarding its deformation and the angle of internal friction. The facing would not participate in the structural functioning of the reinforced portion of the lime fly ash stabilised wall. Thus the dam can be considered to be a composite section consisting of a reinforced zone which is amenable to analysis by limit design methods since it has sufficient ductility and its behaviour is like an ideal elastic strain softening solid. Under conditions of plane stress, the angle of internal friction should be higher than the angle of internal friction under triaxial stress conditions. Hence, there is an in-built reserve of safety if strength parameters are based on results of triaxial tests. Experience of performance of poly-propylene tubing buried in earth dams for piezo-metric measurements indicates that this material is not subject to any biological degradation for a period of 20 years. The system of reinforced stabilised soil, therefore merits very serious consideration for construction of a gravity dam.

The proposed design concept

The dam is proposed to be built of lime fly ash stabilised soil with poly-propylene reinforcement. The proposed system has the following advantages :

—Minimal aggregate processing and transportation cost.

—Placement of material of wet consistancy which makes mixing and placing easy.

—C, ϕ characteristics not very critical with regard to variations in mixing and some variation in lime fly ash content is permissible.

—A wide range of soils can be used to give desired C, ϕ characteristics.

—All grades of lime can be used i. e. fat shell lime to Kankar lime.

Lime fly ash stabilised soil is believed to be free from internal erosion hazard and is reasonably imprevious $K \sim 10^{-4}$ to 10^{-5} cm/sec.

Material Characteristics

C and ϕ characteristics of lime fly ash soil is proposed to be optimised so that cost of lime+fly ash+reinforcement is minimum for the soil available at economic lead. The soil can be a mixture of coarse aggregate and soil matrix. The coarse aggregate would be locally available hand broken and low grade which can be collected without use of any equipment i.e,, the type of aggregate usually stacked in scarcity relief or employment guarantee schemes.

Mixture of fine soil and coarse aggregate will help to minimise lime+fly ash consumption. For example. laboratory studies have indicated the $C \sim 2$ kg/cm^2 and $\phi > 42°$ can be easily achieved with lime content of 60 kg/m^3 & Fly ash content of 60 kg/m^3.

Structural Behaviour

The *zone A* (Figure 23 and 24) is reinforced over the full width. The reinforcement is provided with loops of polypropylene. The *zone B* is also similarly reinforced. Each segment of zone *B* is expected to function like an earth retaining wall. The zone *A* acts like a surcharge which imposes vertical as well as horizontal stresses. Water can also be considered as a surcharge load and the earth pressures acting on zone '*B*' can be computed by active pressure theories. The zone *C* can be considered as a soil mass. The draw down conditions can be analysed by the use of slip circle methods with interslice forces considered according to

Bishop's method. Advanced methods such as Janbu, Morgenstern-Price with non-circular failure surfaces can also be used.

The merit of the proposed structural form is that the design analysis is based on conventional and proven theories which constitute the basis of designs of earth dams, retaining walls and structures in reinforced soil. What is new is a synthesis of soil stabilisation and soil reinforcement concepts.

The following features of the structural system may be noted :

- The polymer reinforcement has a capacity to withstand large strains. This improves the energy absorbing capacity of the reinforced soil system and imparts ductility.
- Hazard of progressive failure is eliminated by virtue of ductility of the reinforcing system. Normal factors of safety say 1.5 can be used for analsis of stability of the soil mass of Zone *C* which is considered as a Mohr-Coulomb material.

The only element of encertainty is regarding the distribution of vertical stress and shear stresses at the interface of the Zone *A* and Zones *B* and *C*.

The basis of design for various alternatives is explained in annexure 9.

Optimisation of Geometry (Figure 24).

The Zone *A* has the same geometry as a hollow gravity dam and the drain is also similarly located. The height of zone *A* will be limited to 20 M. This would limit the maximum vertical principal stress to 5 kg/cm^2. Mobilisation of σ_3 of 0.5 kg/cm^2 would be adequate and reinforcement provided accordingly.

Zone *C* has been given an upstream slope of 1 : 1 since this is expected to be stable without reinforcement. The provision of reinforcement is limited to the anchoring of the facing blocks. This relatively flat upstream slope with a sloping drain will improve the sliding resistance.

Zone *B* is given a down stream slope of 0.4:1 which is a continuation of the upper section and again corresponds to the geometry of the hollow gravity dam For dams of 50 M height, the average lime content is expected to be 40 kg/M^3.

Method of Construction

The technology of the optimum gravity dam aims at minimising the requirement of mechanised equipment and energy consumption in operation of equipment. The volume of material handled will be about 25 per cent of a conventional earth dam. No compaction will be needed and the required density will be achieved by control of grading and admixture of lime and fly ash.

The volume of earth work for medium size earth dams usually ranges from 1 to 4 million cubic meters with conventional design. For the alternative technology, this volume will reduce to 0.3 to 1.0 million cubic meters. Assuming a construction period of 3 years i.e. total of 600 working days the maximum output per day will range between 500-1600 Cu. m.

Past experience of masonry dams e.g. Tungabhadra dam and Maharashtra & Gujarat dams indicates that a daily output of a 1.000 cu.m. can be easily attained with manual loading and unloading and placement at the final position in the dam. Mechanisation would be of the greatest advantage where lifting is involved and for mixing operation.

In the proposed system of construction, fullest advantage will be taken of gravity for haulage of material from the quarry to the dam site. Since the requirement with regard to the characteristic of the earth material (soil and coarse aggregate) are not very exacting, it is possible to locate quarries in the river beds and the flanks so that the required material can be obtained within a distance of 500 meters of the dam axis on the upstream and downstream. The maximum round trip with this haulage distance will be 1 km., the average being 500 meters. One cycle of the haulage unit can, therefore, be completed in about 15 minutes. For a daily output of 500 cu.m. and 12 hour working, the hourly quantity to be hauled would be about 40 cu.m. With four trips per hour, only 10 units of 1 cu.m. capacity are required. This is a very modest contingent of haulage equipment and hand pushed tipwagons can very well be used. The haulage operation can be speeded by using a grade of about 1 in 100 for the loaded movement and the return track can also be provided with a similar grade. A winch can be used for pulling the wagon and for transferring from the forward track (for loaded movement) to the return track (for empty movement). The optimum size of the wagon and the type of trolley track can be developed after some trial. To begin with, the standard 1 cu.m. tip wagon can be used. It may be possible to consider smallar trolleys of 0.3 to 0.5 cu.m. capacity moving on a monorail track with stabilising wheels rolling on a track of Ferro-cement planks. The monorail system has the advantage that sharp curves would be easily negotiated.

It would be evident that the investment in plant would be minimal for the above approach, the life of the plant will be very long and the equipment including the winch, trolleys or wagons can be easily maintained in a small workshop.

Two alternative methods can be considered for haulage and handling along the axis of the dam. In the first alternative, the material can be conveyed along trolley lines placed on the ground or on trestles on the upstream or downstream of the dam. Handling from the trolley line could be carried out by a series of tower cranes located near the extremeties of the base of the dam. Movement along the trolley tracks can be speeded up by providing suitable gradients and the transfer from the forward line to the return line would be by winch ramp. The movement along the axis would, therefore, be by gravity. Alternatively, haulage and placement along dam axis would be by monorail system mounted on trestles.

The trestle piers will be made of the same material as the dam and would be incorporated in the body of the dam. Precured blocks of the lime fly ash stabilised soil would be used for this purpose and these piers would be constructed like brick masonry. A somewhat similar method was used for the Tungabhadra Dam construction where tip wagons were moved on trolley tracks supported by the trestles. The monorail with a

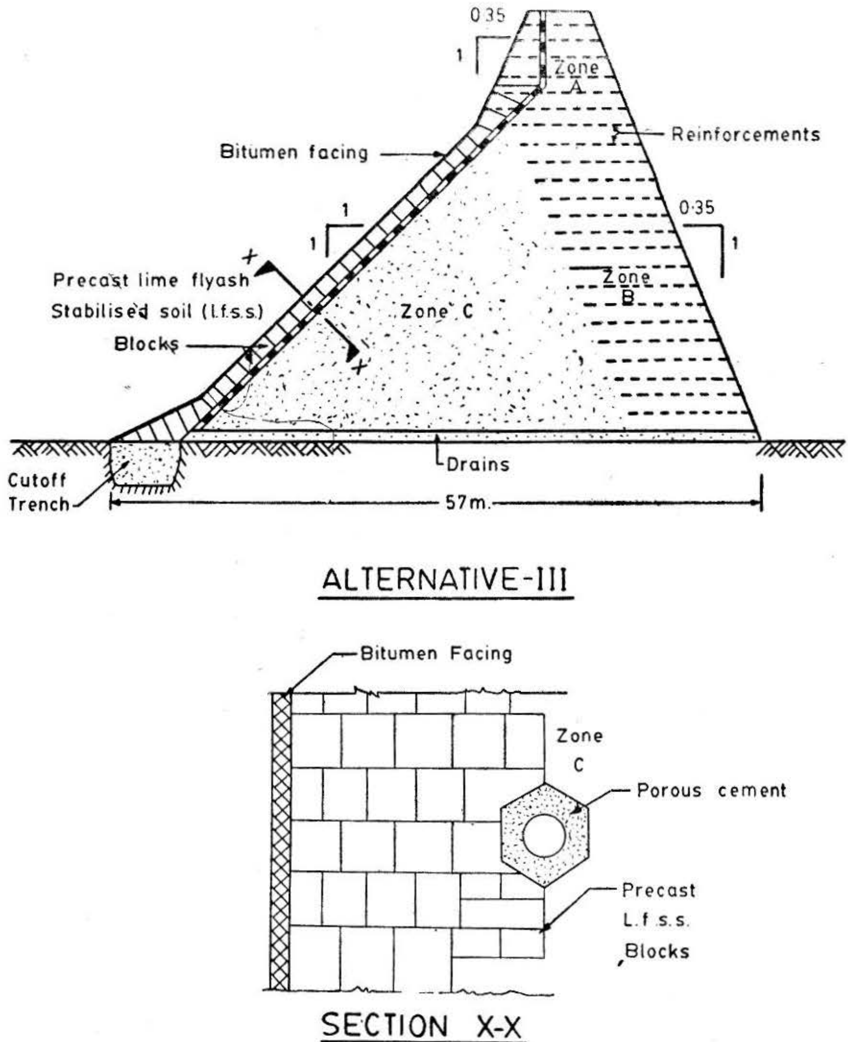


FIGURE 24 Sections of reinforced stabilised soil dams.

diesel drive will have a larger capacity and can be laid out to reach any part of the dam since the grades and curves for the monorail would not be as restrictive as the hand pushed trolley tracks. If desired, a multiple track of the monorail could be used to facilitate distribution of the material along width of the dam so that movement across the width of the dam would be minimised. With this method of handling and placement requirement of the labour force would be very small and since the material is essentially like concrete, very few masons would be required. Skilled labour will be needed only for fabrication and placement of the facing elements and trestle piers. Excavation and loading of the soil and collection of the coarse aggregate could be very well be carried out by unskilled manual labour. The transfer from the trolley tracks bringing the material from the borrow areas to the monorail system for placement at the dam site will be carried out through a system of conveyors, loading and unloading hoppers.

By introducing a set of chutes inclined in oposite direction, the mixing operation can be carried out simultaneously with the handling at a very low cost. A slurry of lime fly ash and a sandy soil or fine sand can be prepared by using a Centrifugal mixing pump like a concrete mixture and this slurry can be introduced into the system of chutes so that the zig-zag movement in the chutes will bring about the mixing of the slurry, the soil and aggregate Figure 25. Since water consistancy is aimed at from the final mixture, the mixing operation should be quite fast. It may be noted here that the system of lime fly ash soil has an ability to tolerate some extent of variation in the proportion of lime fly ash and soil. Very exacting standard of uniformity of mixing are, therefore not necessary.

Energy saving and cost reduction

The results of the comparitive cost study are summarised in Annexure 10. For the reinforced stabilised soil dam, the section according to Alternative II, Figure 23 was considered. This has a section of 600 m^2 and sectional areas for the earth and masonry dams were taken to be equal to 2.7 H^2 and 0.45 H^2 respectively. Where H is the height to the dam in meters. It would be seen that saving in the reinforced stabilised soil dam is about 50 per cent of an earth dam or masonry dam. Since the reinforced stabilised soil dam can be built on relatively deformable foundations, the actual saving as compared to masonry will be even greater. An overflow dam can be built of reinforced stabilised soil this leads to further economy in comparison with the earth dam in respect of cost of junctions and transitions. The facility of passing overflows during the construction would simplify scheduling of construction. The capital outlay on plant will be very small as compared to earth and rockfill dams and the equipment will have a long life, it can be manufactured indigenously and maintained in a small workshop.

The energy consumptions for various alternatives are compared in Annexure 11. It may be noted that energy in rail transporation is neglected. The energy savings are evidently very substantial for the reinforced stabilised soil dam.

Research needs

In view of the potential for cost reduction and energy saving, the proposed system of the optimum gravity dam merits funding of research on a priority basis. The emphasis should be in the first instance on construction technique for which field trials of mixing and handling of the materials could be taken up on medium size dams of 15 to 20 metre height. Theoretical research is required mainly in respect of the behaviour of the reinforced stabilised soil, the structural action of the reinforcement and optimisation of the geometry.

Annexure 9

Basis of design of the reinforced stabilised soil dam

The material for the dam is so selected that there would be no pore pressure during construction and draw down. The steady seepage pore

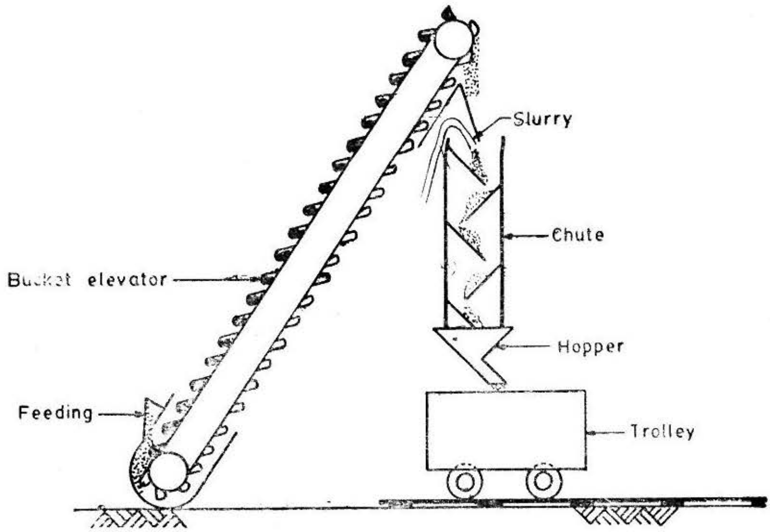


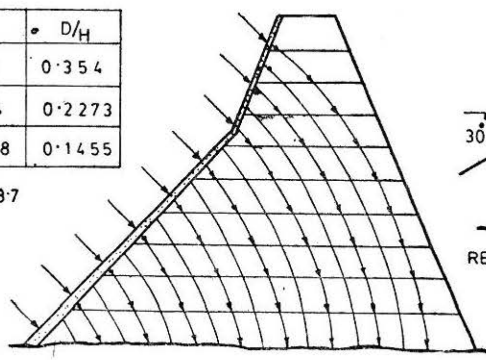
FIGURE 25 Method of mixing

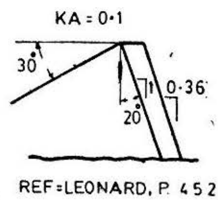
pressures will be brought down by provision of drains. Thus the slopes provided are stable for the soil strength characteristics that can be attained for the stabilised soil.

COMPARISON OF K_A & $D/_H$

Ø	K_A	$D/_H$
30	0·333	0·354
40	0·2174	0·2273
45	0·1598	0·1455

REFER PARA 3·8·7
FIG. 3·6





The down stream slopes is reinforced and is analysed like a reinforced soil wall. Reinforcement provisions would be according to the design methods for reinforced soil but with due consideration of the geometry of the earthfill and the charactristics of the reinforced stabilised soil. As would be seen from the following table, since Ka ia only 0.1 for the proposed geometry a length of reinforcement of about 3×0.1 of the height would be adequate for a soil of 'ϕ'$=40$

Another criteria for the reinforcement design would be to consider the reinforced soil to behave like an assemblage of columns. The geometry is so arranged that the forces due to water pressure will create mainly axial forces as seen from the figure. The results of triaxial tests can thus be used as a basis for designing the reinforcement. Both the above criteria should be satisfied.

Annexure 10

Cost of Alternatives per M. for 30 M high Dam

Item	Lime flyash stabilised soil			Earth Dam			Masonry Dam		
	Rate	Qty/m	Cost	Rate	Qty/m	Cost	Rate	Qty/m	Cost
1. L.F.S.S.	54/-	600/m	32,400						
Reinforcement	20/-	135 Lit.	2,700						
2. Earth work	—	—	—	20	2415	48,300			
Riprap rock toe, etc.	—	—	—	8	2415	19,320			
3. Masonry	—	—	—	—	—	—	200	360	72,000
			35,100			67,620			72,000

Stabilised soil unit rates

	Alternative I			Alternative II		
Iteme	Unit Rate	Requirement/m³ kg/m³	Rate/m³ Rs/m³	Unit Rate	Requirement/m³ kg/m³	Rate/m³ Rs/m³
Lime	300/T	60 kg/m³	18/m³	500/T	36 kg/m³	18/m³
Fly ash	100/T	60 kg/m³	6/m³	200/T	36 kg/m³	7.2/m³
Mixing Cost/ placing	—	—	6/m³			6/m³
Earth work cost	—	—	24/m³			
Rubble and low grade aggregate				20×0.4		8/m³
Earth including mixing				25×0.6		15/m³
	Total	54/m³			Total	54/m³

Thus overall rate of Rs. 54/m³ can be realised for a range of lime flyash/pussolna,

Annexure 11

Energy component R/s meter of dam

Reinforced lime flyash stabilised soil (LFSS) Dam:

Energy component 75 per cent of cost	Rs. 9,600/-	
Energy component for mixing, handling and placement 15 per cent of earth work cost	Rs. 2,700/-	
Polypropylene reinforcement	Rs. 2,700/-	Rs. 15,000/-

Conventional Earth Dam:

Earth work energy component 60 per cent of cost	Rs. 40,572/.

Masonry dam in cement mortar:

Energy component 70 per cent of cost	Rs. 56,700/-

Energy for transportation

	L.F.S.S. Dam	*Earth Dam*	*Masonry Dam*
Cross Section	0.8 H^2	2.8 H^2	0.4 H^2
Unit weight T/m^3	1.8	1.8	1.8
Loads for soil stone or sand	0.5 km.	1.5 km.	6.0 km.
Load of lime or flyash/cement	20 km.	--	40 km. Cement
Haulage per M of dam	2,7 km/H^2	7.6 Tkm/H^2	6.5 T km/H^2
Diesel for 30 m dam	121.5 1 it.	342	292.5

Rail transportation not considered.

Requirement of reinforcement

Mean volume proportion of reinforcement works out to be 0.5 lit/m² based on allowable stress 1,000 kg/cm² in reinforcement and taking average volume proportion as 2/3 the maximum volume proportion.

Concluding Remarks

I would like to conclude with a few general observations. The design concepts and construction techniques described in this lecture represent an approach to technology which aims at overcoming resource constraints. No finality is claimed for the theoretical formulations and some of the structural forms may be radically modified after field trial. In 'research and development' there should be a major element of field trial and performance evaluation.

I realise that participation of many others, who are better qualified to undertake the theoretical studies and development of equipments, will be needed to perfect the suggested approach to soil improvement. This contribution would have served its purpose if it can usher a new approach to research and development in geotechnical engineering, which would be relevant to India and other developing countries.

Acknowledgements

I would like to thank the IGS Executives for making a unanimon decision to invite me to deliver this lecture. I take this as a manifestation of the continuing interest of the IGS and particularly Prof. Dinesh Mohan, past President and Prof. Jagdish Narain, President, for their personal interest in my research work in soil improvement.

At the outset, I would like to express my feeling of gratitude to the outstanding engineers in India and abroad with whom I had the privilege of working in the early stages of my professional life. More specifically, I would like to mention late Mr. G. G. Dhanak Chief Engineer, Gujarat State; Mr. N. G. K. Murti and Mr. G. N. Pandit, former Chief Engineers in Maharashtra State. Whatever professional skills I have acquired are entirely due to their encouragement and guidance.

An opportunity to study French practice of alluvial grouting and diaphragm walls in 1960, under the guidance of Mon. Armand Mayer has left a deep impact on me. This helped me to understand and emulate the French approach where outstanding innovations have been made on the basis of simple theories perfected by model study, field testing and prototype observation.

The evolution from concept to design and field application was made possible through the help and assistance of associates and engineers with whom I have been working in the last 14 years of my practice. Comments and suggestions of Mr. P. R. Tongaokar, Dr. C. M. Pandit and Dr. N. G. Bondre have been especially valuable. It is not possible to acknowledge all those who helped me, individually, but I would like to mention Mr. S. B. Bhide, Mr. A. V. Gogate and Mr. R. D. Chouthe. Engineers, for their contributions to analysis, field control and monitoring and their perseverence and dedication. In the laboratory work and analytical studies I have been ably assisted by Prof. S. S. Nagaraju and Mr. V. N. Gore. *Mr. Gore also helped in overall coordination of research activities.*

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The credit for successful application of the ground improvement technique, particularly stone columns and 200 mm sand drain, is due to the support from my clients. I would like to particularly mention :—

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