SYMPOSIUM ON PORE PRESSURE AND SHEAR RESISTANCE OF SOILS, MAY 67, NEW DELHI.

EXPERIENCES IN SHEAR TESTING FOR PROBLEMS OF EARTH DAM FOUNDATION AND EMBANKMENT MATERIALS

BY

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SUMMARY

A review of shear testing techniques has been described. Shear Characteristics of highly plastic clayey soils under natural and remoulded conditions along with correlations of shear parameters (C' & Ø) with void ratio for undisturbed samples have been illustrated. In general, test results have been found to be higher than values recommended by various foreign standards.

It is seen that there is reduction in Ø for higher range of confinement pressures for clayey and silty soils which points to the necessity of triaxial shear testing being done under appropriate range of confinement pressures. The need of reduction of rate of strain with increasing confinement pressure for correct assessment of pore pressure and volume change is brought out by the experimental results.

INTRODUCTORY

The difference between soil mechanics in theory and practice has been increasingly felt since the introduction of subject in the country. The practices used by various laboratories and design offices in Government have indicated more conservatism and resistance to adopt improved new but untried concepts. This is but natural as the subject involves materials which are rarely uniform in texture and behaviour under different conditions. In order to understand the practice which has stood test of time, study of history of organisation is essential.

HISTORICAL DEVELOPMENT

Necessity of feasibility studies for the irrigation and power projects with reference to construction materials and foundation problems led to creation of Directorate of Silt & Construction Materials under Central Water & Power Commission in 1954. The subject of construction material and foundation investigation was considered mostly a subject of routine testing and sampling which can be handled by scientists. The approach was, more or less, scientific and limited to feasibility studies. Gradually, for implementation of plan schemes, problems of final designs and construction prointed to the need of more emphasis on the engineering outlook. This and introduction of soil engineering subject at University level in 1960 brought 'Soil Mechanics to action' in India. Special United Nations aid to equip the Directorate to carry out investigations of 62 hydro-power schemes led to expansion of teh directorate to a fullfledged Research Station named as "Central Soil Mechanics Research Station". The function of the Research Station include providing technical assistance and advice on the problems of soil mechanics and foundation engineering encountered during investigation, design and construction.

Earth dams as old as 800 years are in this country, but scientific approach to design and construction was introduced with Gangapur Dam (Bombay) and Hirakud dam in Orissa. Use of highly plastic clays in homogenous section is high light of most of old dams. Shear parameters obtained by quick direct shear test (till 1958) had resulted in low angle of internal friction and high cohesion and wide variation of results. The design practice was to select an average low value and reduce it to 66% for adopting in total stress analysis. The pre-determined plane of failure and uncontrolled drainage condition led to wide variation of results in direct shear. This resulted in development and introduction of triaxial shear test all over the world. This was introduced first in Central Water and Power Commission in 1957-58. The tests carried out were undrained quick shear with the time of failure of 10 to 15 minutes. Effective shear parameters concept was introduced in 1962 when pore pressure measurements were made during triaxial shear test. Though the introduction of effective stress concept was lagging behind by about 20 years, it could not be adopted for designs directly due to the difficulties of assessing construction and drawdown pore pressures etc. The development since 1962 includes introduction of larger specimen sizes 4 to 6 inches diameter higher pressure ranges for lateral confinement upto a maximum of 2000 psi, wide range of application of rate of strain, proving rings up to 10 ton capacity, pore-pressure measuring devices of various types - Imperial College, Building Research, London and Transducer for pressure upto 200 psi.

TEST PROCEDURE

Tests are conducted on undisturbed samples from foundation and remoulded samples for construction material. Coarse porous stones (bubbling pressure < 2 psi) are used for measuring pore water pressure. The time of failure has now been kept at 2 hours minimum and longer duration of 4 to 5 hours have been resorted to in some cases.

Foundation samples are tested usually of 1½ inch diameter at natural dry density and natural water content, though saturation by keeping *samples in water for 8 to 10 days have been attempted somtimes. The test on the foundation samples may vary from undrained to consolidated undrained conditions depending on state of saturation, depth of samples and type of samples. In partly saturated samples, capillary pressure (Uc) will exert all round compressive stress and release of effective normal and lateral stresses due to sampling will create tendency to expand. Volume change must occur for equilibrium to be established. Undisturbed sample will have no shear stress on any plane identical to insitu state. Unconsolidated undrained tests are normally applied to partly saturated samples of low depths. Consolidated undrained tests are used for deep, saturated samples to assess correctly the shear stress at different effective normal stress. Normally consolidation of 24 hours is applied under all round pressure. Typical plot of test data is shown in Fig. 5A and 5B.*

Construction material samples are usually remoulded to proctor dry density at saturation moisture and covered by filter strips for quick dissipation of pore-pressures. Tests conducted now are mostly consolidated undrained and some times drained tests for previous soils.

SHEAR PROPERTIES OF CH SOILS

<u>Classification of soils:</u> Highly plastic clay forms a major part of the earth crust in most of the high rainfall areas in our country. Some of them are predominant montmorilonite whereas the others are lateritic. These soils exhibit tendency of reduction in shear strength on wetting and swelling on saturation. Undisturbed samples are very difficult to saturate due to low turbed samples are very difficult to saturate due to low permeability thus making interpretation of tests more difficult. Some tests of the typical highly plastic clays that have been tested during the last two years have been analysed to assess range of shear parameters.

The clay content of soils range between 30 to 53%. The liquid limit is greater than 45 to 50 whereas plastic limit range form 20 to 30 percent. The natural water content of these samples is found to range between 16 to 32 percent for void ratio range, 0.6 to 0.9 and saturation greater than 75 percent.

<u>C' - ∅' relationship for undisturbed state:</u> The above relationship for saturation range of 75 to 100 percent and plastic limit range of 20 to 33 percent, the graph of void ratio against effective angle of friction (∅) obtained by triaxial shear test is shown in Fig. 2. It also incorporates Russian and the USBR standard values (Design of small earth dams). It will be seen that there is a very clear trend of decrease in effective angle of internal friction with the increase in void ratio at higher rate than stipulated by the Russian standards. On an average ∅' decreases from 27° at void ratio of 0.65 to 14°at a void ratio of 0.95. It is expected form the trend of the graph that with

the increase in the void ratio, no further reduction will occur in ϕ'. The lowest value compares favourably with the Russian Standards whereas the difference increases as the void ratio decreases from 0.9 to 0.6. USBR shows $\phi' = 28.5$ at $e = 0.56$ whereas our result indicates on an averagge angle more than 28°. Hence for all the practical purposes, the results obtained on undisturbed samples indicate higher values of angle of internal friction for the Indian highly plastic clays under effective stress conditions. Clayey soils having liquid limit > 65 and PI > 30 rd < 1.38 g/cc and NWC > 30 percent have exhibited $\phi' = 0$ by present method of testing.

<u>Effective value of cohesion in undisturbed state:</u> Effective cohesion (C') is plotted against void ratio in fig. 3. For the range of soils tested upto a saturation of 90 percent, there is a tendency for the cohesion to decrease with increasing void ratio for the full range of plastic limit. On an average, cohesion decreases from 0.45 to 0.1 Kg/cm^2 for the range of void ratio 0.6 to 0.9. For the saturation more than 90% very steep reduction in cohesion is observed. An attempt made to find out the effect of saturation on cohesion for given plastic limit range did not show any relationship except that the increase of saturation more than 80 percent has little or no tendency to reduce cohesion further.

<u>Typical Mohr circle envelopes for undisturbed state:</u> The average C' and ϕ' values for undisturbed highly plastic clays obtained from different projects all over the country have been plotted in Fig. 1, for total and effective stresses under unconsolidated undrained conditions.

The angle ϕ' under undrained condition and total stress is 7° to 24° as compared to corresponding range of 21° - 29° for effective stresses.

<u>Shear properties under remoulded conditions:</u> It has been found that materials of construction of core are similar to those of foundation, both of them being highly plastic clays in some of the projects.

The soils are remoulded at saturation moisture content to the proctor's density and tested by triaxial under undrained as well as consolidated undrained condition. Fig. 4 (b) is plotted to show variation of total stress shear envelop, whereas Fig. 4 (a) shows corresponding effective stress envelops. The range of undrained test indicated lowest value of ϕ' as 7° and highest value as 28° whereas corresponding range for consolidated undrained conditions is 14° to 26°. The range of effective angle of internal friction(ϕ') is 18.5 to 32°. Consolidation and pore pressure measurements have considerably reduced the range of values. Remoulding was carried out at saturation moisture content and it was expected that the arrangement of the particles will be nearly normal to deviator stress. In most of the remoulded soils, the strain at failure was more than 15 to 20% whereas for undisturbed samples it was less than 15%. This may be explained by difference in undisturbed and remoulded structure of samples.

It can be stated that under the identical conditions of testing, the effect of remoulding has practically little effect on shear parameters at maximum deviator stress. The variation in the values may be due to difference in state of manner of deposition, consolidation, desiccation and variation in mineral composition and grading of soils.

EFFECT OF HIGHER CONFINEMENT PRESSURE

The usual practice is to test specimens up to maximum lateral presssures of 60 to 80 psi. With introduction of higher earth dams, effective confinement pressure in prototype will be much higher then the range stated above. In order to study the effect of higher confinement pressures on the shear parameters, soil samples of ML group from Beas Project were tested up to maximum confinement pressures of 250 psi. These test results have been illustrated in Fig. 5A/5B. Confinement pressures more than 160 psi reduce ϕ considerable with the corresponding increase in effective cohesion. Similar test on SH group of samples do not indicate such trend.

The preliminary tests for Beas soils indicates teh necessity of revising the concept that C - ϕ are constant for fall ranges of pressures for clayey and silty soils. The trend of pore pressure development and volume change as well as deviator stress indicate marked difference in pattern for pressure more than 80 psi. Adoption of different values of C' and ϕ' for different heights of dams will have to be introduced for high dams to make design more realistic. Further studies for high pressure behaviour are in progress.

THEORETICAL VERIFICATION OF PORE PRESSURE

Techniques of measurement of pore air and water pressures are still imperfect. U.S.B.R. has evolved special low air entry porous ceramic stones which do not permit transmission of air into pore pressure measuring system. In a partly saturated soil, total pore pressure will comprise of pore air pressure and capillary pressure. The coarser type porous stones which are in use at the present will be measuring Ua for soils having fairly low moisture content on Ua + Uc if moisture content is sufficient, Uc being negative, the results obtained will vary considerably depending upon moisture content of specimen and end conditions. Therefore, Hilf (1956) recommended theoretical equation of $Ua = \dfrac{Pa \times \Delta V}{VA + hv_w - \Delta V}$ to check laboratory pore pressure measurements. In order to apply this equation, a study was made on the ML type of soils samples from Beas. The theoratical curves and actual pore pressure observations are shown in Fig. 6.

It can be seen from the results of 15,30 and 45 psi pressure conform to values obtained theoretically. Test at 60 to 90 psi at which larger amount of air has gone in solution confirm to theoretical curves of

Va + hvw = 5 to 7. The rate of strain used in faster thereby under-estimating pore pressure at a given strain. This method has been very useful for arriving at rate of strain for the test by verifying volume change and the pore pressure during test. It also indicates necessity of slower rates of strain for higher confinement pressures.

INSITU SHEAR STRENGTH

In very soft saturated clays, it is very difficult to carry out undisturbed sampling to asess shear strength of the soil. In such cases, the problem is tackled by in-situ measurement of shear strength by using vane shear test. The equipment is to Norvegian design with controlled rate of application of torque. The assembly is pushed by jacking into subsoil. The vane is then pushed out of the protection shoe to a depth of four times the diameter of the shoe. Torque at the controlled rate is applied to shear on a cylinderical surface of soil subjected to effective overburden pressure. This Norvegian procedure has a definite advantage in eliminating drilling, dummy test, lowering and withdrawing of vane for each test. Investigations were carried out by the Research Station for the Marine clays in little Rann of Kutch and Bombay and have indicated the following additional advantages:-

1) The effect of partial removal of overburden by drilling is eliminated which contributes to increase in the strength 50 to 100% depending on type of soil, sensitivity, depth etc. Thus the strength obtained by above equipment climinates an under-estimation of strength due to testing procedures.
2) Time required for conducting the test is reduced considerably.

The details of the studies carried out by vane shear test on marine clays are being published in a paper on 'Foundation Investigation of Marine Clays' at 3rd Regional Conference, Israel.

TRIAXIAL TESTS ON ROCKS

The results of shear test done on Tawa sandstones are plotted in Fig. 7. Tests have been carried out under higher confinement pressures. Effect moisture absorption on ϕ is shown in the figure.

CONCLUSIONS

1. Review of recent test results on CH soils in India indicates the that value of ϕ' is comparable to or higher than USSR and USBR standards.

2. Necessity of study of shear envelope for high dams for fine grained soils is felt due to likely reduction in ϕ' and corresponding increase in C'.

3. *The measurements . of pore pressure should be verified by theoratical calculation in order to eliminate error in measurements.*

4. *Higher the confinement pressures for given soil, lower rate of strain will be required for correct assessment of pore pressure.*

ACKNOWLEDGEMENT

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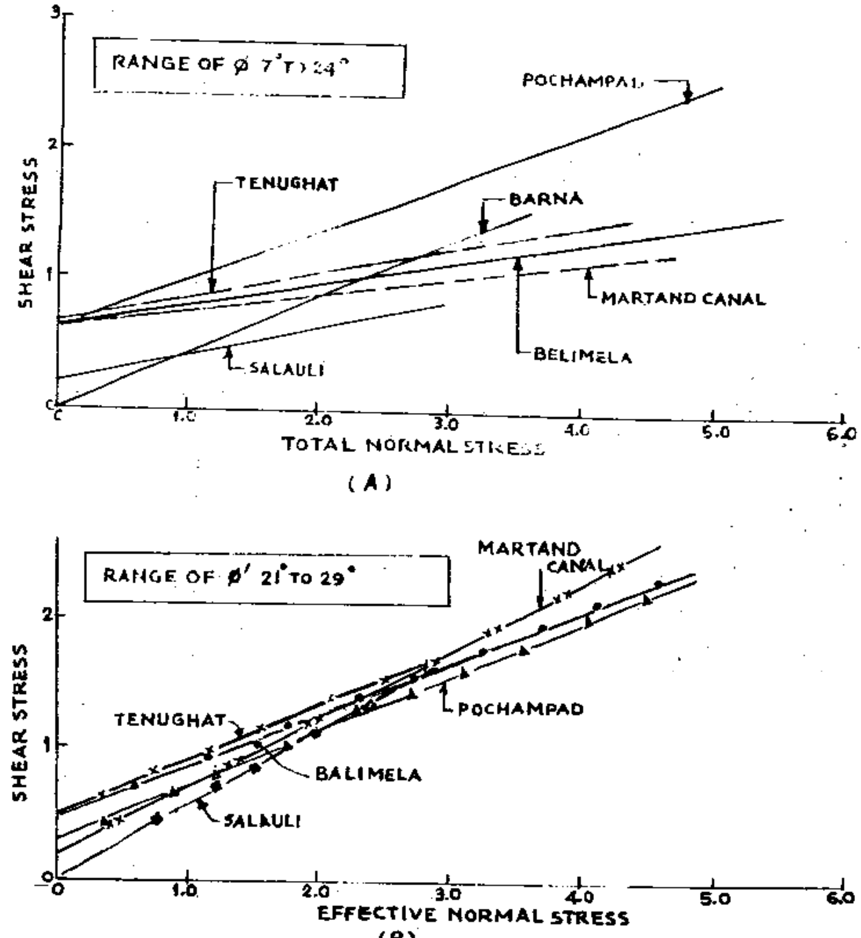

FIG. 1 SHEAR TEST ENVELOPES FOR UNDISTURBED C.H. SOILS (SAT. > 75%)

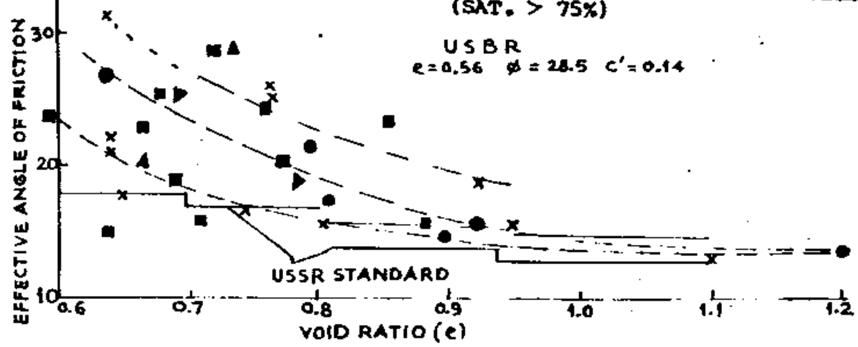

FIG. 2 RELATIONSHIP BETWEEN e and Ø FOR CH UNDISTURBED SOIL SAMPLES (SAT. 75% TIME TO FAILURE 150 Min.)

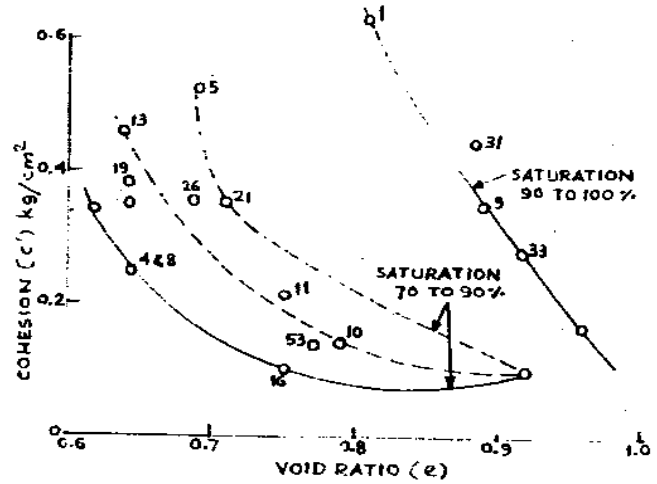

FIG. 3 EFFECT OF VOID RATIO ON EFFECTURE COHESION (C') FOR UNDISTURBED CH SOILS

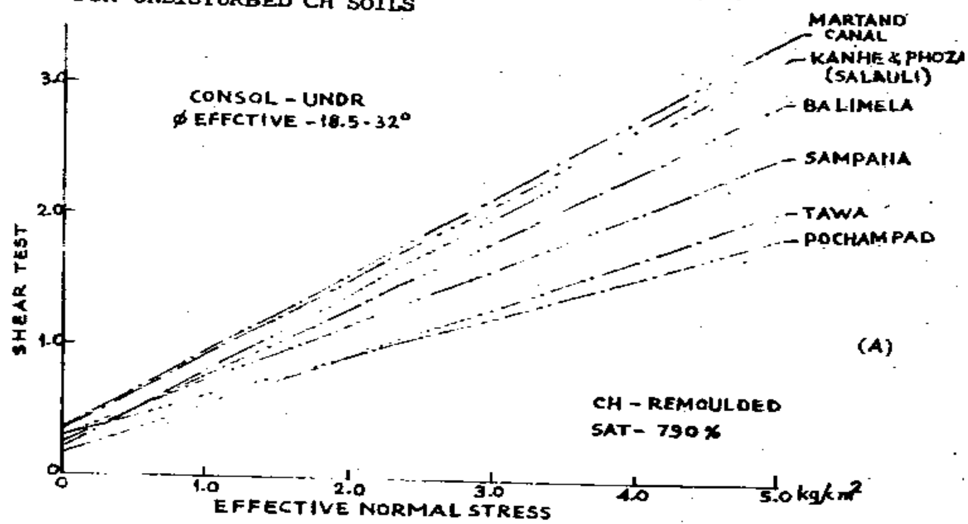

FIG. 4(A) SHEAR ENVELOPES FOR REMOULDED CH SOILS (SAT. 78%)

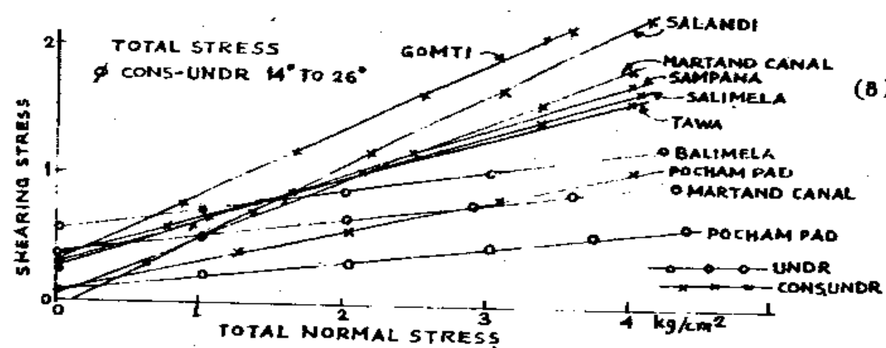

FIG. 4(B) SHEAR ENVELOPES FOR REMOULDED CH SOILS (SAT. 78%)

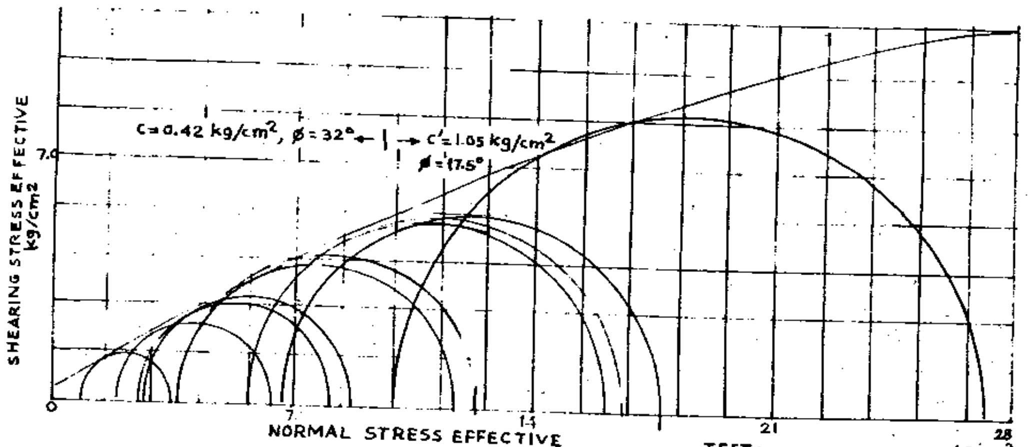

FIG. 5 MOHR CIRCLE
FOR TRIAXIAL TEST ON ML SOILS AT HIGH CONFINEMENT PRESSURE

TEST :-
CONSOLIDATED UNDRAINDED
1).004"/mm
2).OMC

ML = SAMPLE
OMC = 12.90 %
MDD = 1.8859 g/cc

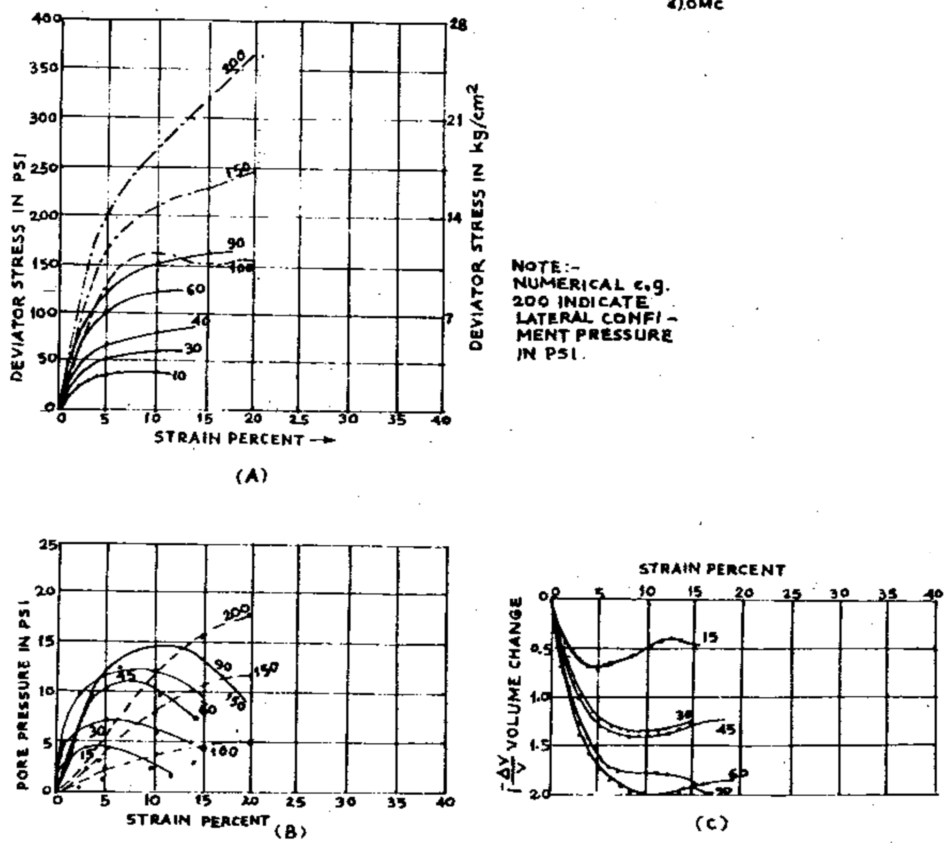

FIG. 5 TYPICAL TEST RESULT OF TRIAXIAL TEST UNDER HIGH CONFINEMENT PRESSURE

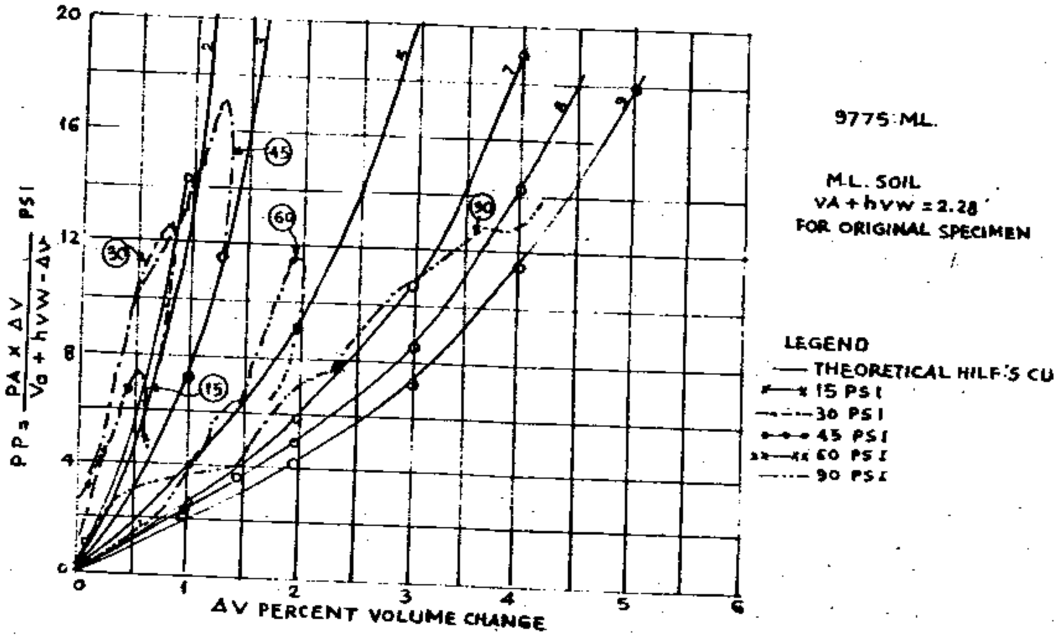

FIG. 6 PORE PRESSURE ACTUAL AND CALCULATED BY HILF METHOD DURING SHEAR TEST

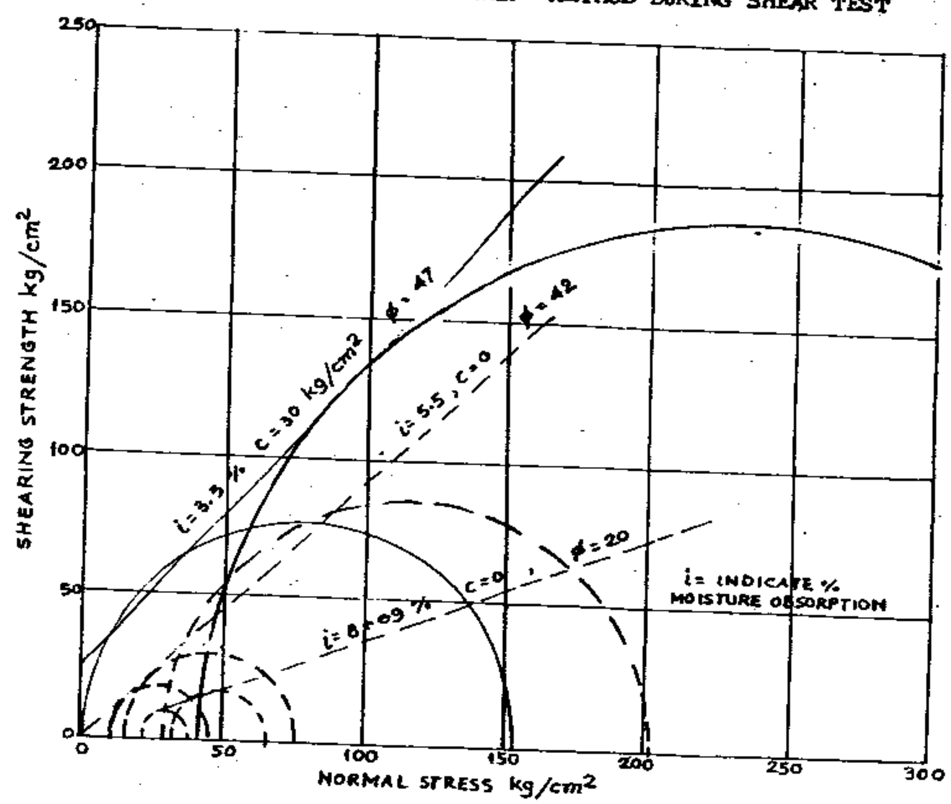

FIG. 7 TRIAXIAL TEST ON SANDSTONE OF TANA

Exploration of Embankments to Highlight Field Studies

M D DESAI

SUMMARY:

Earthwork engineering even now is an art aided by the judgement acquired by use of soil engineering. The present organisational setup generally based on devided responsibility, has been blamed for crisis in Earthwork engineering. To save the situation, need of a common experienced soil engineer responsible for all stages and need for a specialized observation cell have been brought out.

Over emphasis on design and inadequate attention to field observations have led to avoidable troubles. General conviction that all is well if Proctor's density is achieved by excellent density control, is incorrect. It has been shown that a oversafe conservative design satisfying usual field control has not eliminated probability of failure. Soil behaviour has to be watched continuesly through out.

The dam design is based on tests of laboratory compacted samples. The field compaction by sheepfoot roller may not give same structure and hence the strength. A study made here showed that the effect on c' and Ø' are different for CH soils of shalandi and SM soils of Beas dam. In general there is no extra safety factor need be justified on this account.

A similar illustration of Jund - Kandla Rail link is used to emphasize need for field tests. The rate of settlement and total settlements were overestimated by usual computations for this project.

1.0: INTRODUCTION:

Wide gap between need and availability of Irrigation and power, and the need of creating mass employment for semi-skilled workers and shortage of cement presents an excellent picture of feature for earthwork engineering. About 65% of the dams built in the recent past in India and world are earth dams. Till 1960 earth dams of even 60 m was considered as a rare feat of planning and design. In the following 14 years advancements of techniques and confidence gained in earth dam technology can be inferred from trend of the maximum height of the dams shown in the Fig.1.

2.0: ORGANISATIONAL SETUP:

2.1: General: The increase in the cost of the project and delay in its complition are common features of many projects in developing countries like India. It has been atributed to inadequate investigation and analysis in time (Prakash-72). Change in the design, irrespective of any amount of initial explorations has been reported to be inevitable to suit the site conditions, to economise or to expedite construction (Murthy etl-73). Most of the major projects of Irrigation and power are controlled and executed by the Government. Generally such departments have an organisation based on devided responsibility and localizing credit pattern.

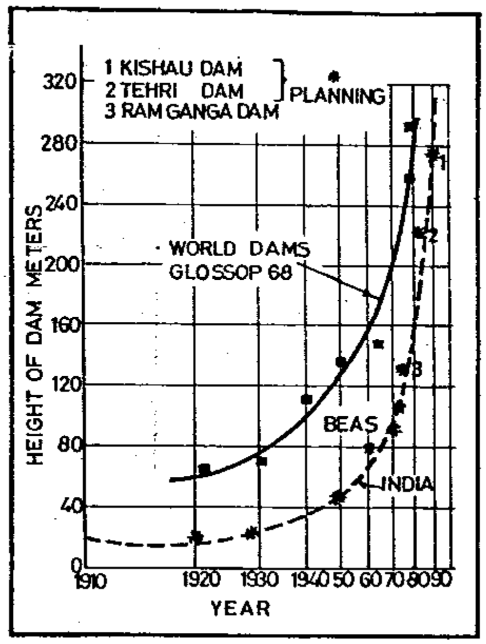

Fig.1: Maximum height of earth dams from 1910 - 1970.

2.2: Field explorations: A preliminary project report based on a similar structure and meger exploration is prepared, for the planning commissions approval, by non specialized general PWD office in that region. It takes years to get final administrative and technical clearance. The final explorations are then taken up by the construction division inexperienced for such job. The staff is generally unaware of availability and relative merits of the tools and techniques for exploration. They have hardly any concepts time needed and critical design parameters for the given project. Being ignorant of logic and line of attack they take care of quantitative progress mostly based on expenditure incurred. Reporting Split Tube samples as undisturbed, driving Sp test tool without predrilling a cased hole, taking undisturbed samples in pieces of thick casing pipes without a shoe of a ball check valve in lengths of 1 metre and providing only coarser fraction after washing out of the fines by the method of drilling like wash or shell boring below water table as true subsoil sample etc. are not uncommon.

2.3: Laboratory testing: The samples thus collected are dispatched to a soil testing unit of state or central research cell. Sometimes they are received after months of collection in dry condition. They never carry any details of the problem, design requirements or the instruction for the test. In one case tender specications for the testing state that all relevent test and possible tests under all variable conditions be carried out. The research cells are subordinated to the design cells. The latter can only critize but rarely guide. Thus to avoid any responsibility a wise research officer will manage some tests under some conditions (of course, they will never be reported) and compete for progress. As he cannot blame field nor design cell hence orders to report a result of undisturbed sample tested by remoulding (as undisturbed) are issued. Particularly when analysis is not their responsibility, the work can be taken lightly. The things gets still worst with the frequently changing staff irrespective of apptitude and liking for such job. The percentage of significant information, with such organisation, can be zero to not more than 50 percent, even with excellent efforts.

The data from the field and laboratory are compiled by officer in charge of construction for onward transmission to design cell through proper channel hardly contributing any thing. This jungle of data, to a novice designer, will be more confusing then assisting. To pose as expert, he will, after taking remarks, finalize designs taking special safety factors for the inadequate self confidence, little confidence in testing, sampling and anticipated poor execution. With this poor basis, it is a fashion to adopt latest techniques and tools like computers to exhibit modernisation of the design cell.

2.5: Construction planning and Observation: Mostly the construction details are rarely accompanied by the design logic and field staff is competent to make minor changes which will infact violate basic phylosophy of design. Such modifications are at times innocent or motivated. Field staff feels control and instrumentation a hindrance to their output. With such cooperation most of projects have been instrumented for name shake only. Added to this instalation by untrained staff and observation by juniors aimlessly and irregularly and too late analysis by other staff for more or less acedemic purposes have brought out logic to ommit them altogather or plant them at few sites with direct control by specialized cell at the center.

2.6: Crissis in soil engineering: All the specialists involved in above stages gets divorced form the project moment they complete their stage. Practcally none is concerned with soil aspect of project in entirity and its impact on overall cost. The opportunity of reducing gap between thory and practice is lost. To add to this more than three officers must have changed hands in each stage as the cadre is transferable. Thus inspite of number of projects completed and progressive increase in height, the present organisational set up has failed to provide real exprerts devoted to the field of earth work engineering. Vast experience has been broken up in pieces and distributed to too many such that it can never be assembled usefully.

Real expert as defined by Terzaghi (59) is one who has a combination of experience, keen sight for observation and insight into the performance of the structures. To achieve such experness effort alone is not enough. It requires innate qualifications over which one has no control (Terzaghi 59). Activities of those acting as expertshave created crisis in earth work engineering. Designs using soil mechanics have become uneconomical to old empirical designs. Failure of the set up has been passed on to soil engineering by stating it to be premature science.

2.7: Remedy: Unless the setup of the organisation is overhauled and a strong central specialized reserch cell with status and non transferable specialists is given entire responsibility of handling all stages from the initiation to study of behaviour, situation cannot improve. Only those having apptitude and devotion to the field shall be recruited and delegated all the powers to deal with earthwork component of the project.The past experience, new trends of design as you observe and increasing cost of earth work justify a thorough overhaul of the present pattern at an earliest. The new set up will act as data centre and take earth dam technology back in the progressive direction. Compared to the present waste, cost of a new cell will be negligible.

3.0: NEED FOR FIELD OBSERVATIONS:

Soil Mechanics is a suppliment to and not a substitute for, common sense combined with knowledge acquired by years of experience.Engineer has to use his brains all the time even if knows theory by heart.(Terzaghi 59).The concept that a conservative design with a rigid control of density is adquate for avoiding a disaster is incorrect. Soil mechanics has to be in action through out by watching every step of execution. This is illustated by an example.

A composite 52 m. high and 818m.long dam involving 3.81 million cu.m. of earthwork, section shownn in fig.2, has been partly completed. During construction in 1965 sampling was done by a contractor to assess permeability of foundation soil. Results of this samples for shear test led to revision of design of down stream slope of dam.To correct design parameters a programme of drilling and sampling at 10,70 and 137 m. down stream at two cross sections was executed.Dry drilling or sampling at least in insitu condition was aimed. 32 foundation and 92 embankment samples were collected with Shelby tube samplers of 37.5 mm diameter.

Analysis of the embankment soil samples and range of plasicity are shown in fig.3. The impervious zone is made of CH soil. The field variation of tube density and water content at a typical cross section are illustrated in fig.4 41% samples showed a dry density range 1.65 to 1.7 g/cc and rest showed a range of 1.7 to 1.85 g/cc. The corresponding Proctor's density range was 1.6 to 2.1 g/cc. Except for few dry pockets e.g. at RL 222 the field moisture is 14 to 20% giving 65 to 92% degree of saturation.

The results of the triaxial tests

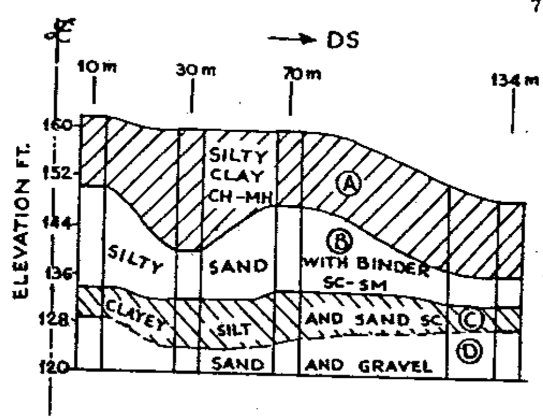

Fig.5: SOIL PROFILE OF DAM FOUNDATION

under different conditions shown in fig 5 led to a recomondation of c'=0.3 kg/cm.sq. and $\phi' = 25°$.

Fig.5 indicated foundation profile with the subsoil properties.Thus above details and design factor of safety by no chance indicate a disaster for this dam.

TABLE-1

Layer	Soil	r_d g/cc	w %	c' Kg/sq. cm.	ϕ' deg.
A	CH	1.55	22-31	0.07	19.5
B	SC-SM	1.52-1.92	14-27	0.3	27
C	SC	1.5-1.67	20-25	0.07	32
D	SW	1.8	-	0.07	40

Though the instructions were to collect samples only, keen observation of abnomal loss of water e.g. at different levels in down stream has brought out abnormality. It was not a stratum nor a layer in cross section. At one place coefficient of permeability was 10^{-1}cm/sec. and loss was continueous indicating that it cannot be a pocket. The water content of this layer indicated low value. (fig.4). Field observation, period of construction, and details noted above hinted at a probability of deep rain cuts formed on upsteam and down stream. Path of such rain cuts on non dressed slopes is zic-zag. The specifications to strip surface before taking up earth work after monsoon might have led to filling of deep cuts by dry clods of CH soils. The clods are practically metal aggregates and have large cavities to permit percolation. It was impossible to locate number or

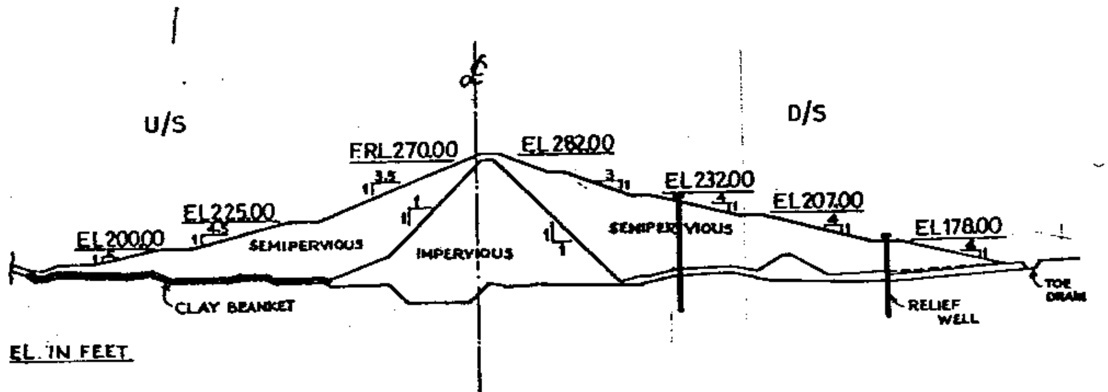

FIG.2-SECTION OF DAM.

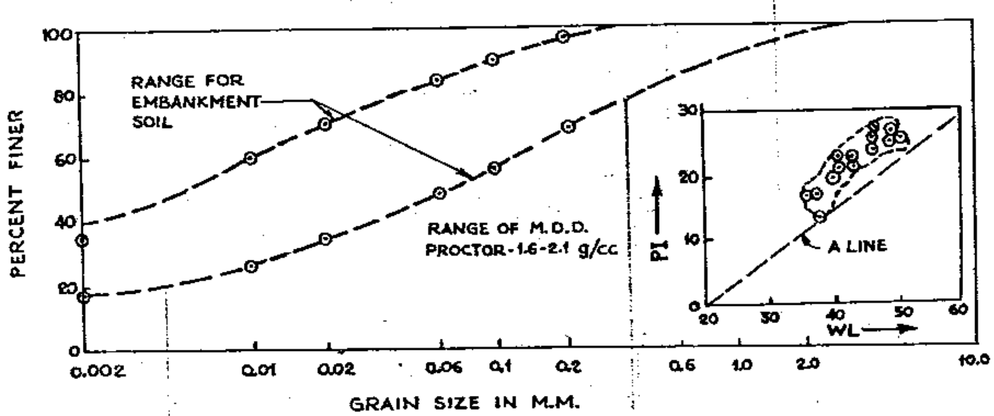

FIG.3-RANGE OF GRAIN SIZE & PLASTICITY OF EMBANKMENT IMPERVIOUS SOILS.

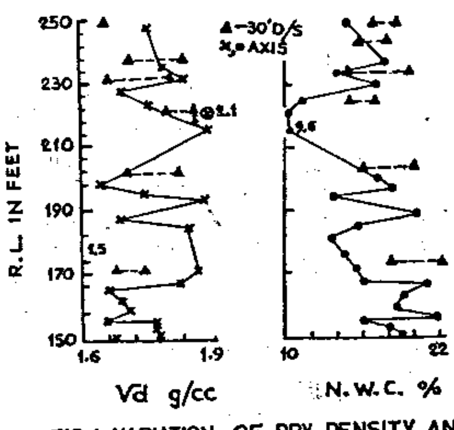

FIG.4-VARIATION OF DRY DENSITY AND HOISTURE CONTENT IN CORE.

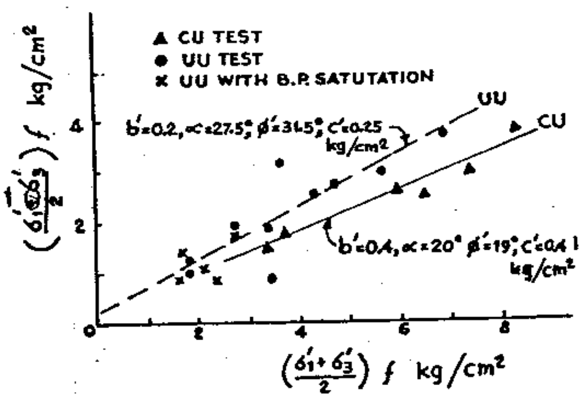

FIG.5-SHEAR STRENGTH OF THE CORE MATERIAL.

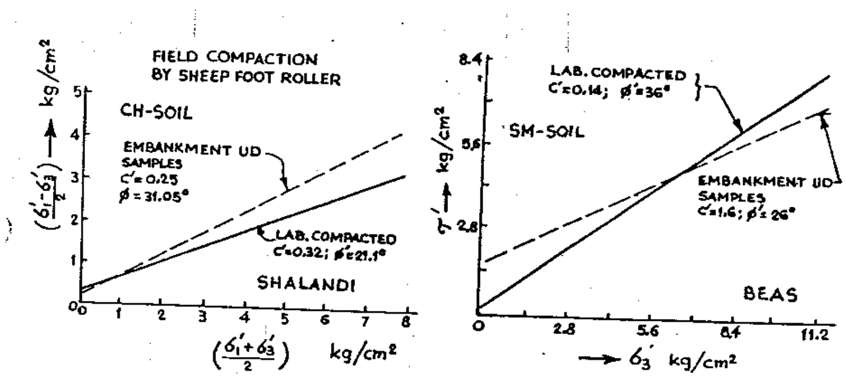

Fig.-7: COMPARISION OF EFFECTIVE SHEAR PARAMETERS OF EMBANKMENT SOILS COMPACTED BY LAB. TECHNIQUE AND BY SHEEP FOOT ROLLER.

path of such cuts and then treat them by grouting etc. Otherwise a safe structure may have met with a disaster due to seepage on filling of reservoir. The timely warning by common sense and keen observation led to slow filling of the reservoir and constant need of vigilance. It was expected that clods will absorb water and expand to become water tight by itself.

4.0: DIFFERENCE IN TECHNIQUES OF COMPACTION:

The practice of remolding borrow pit soil to prepare specimens for triaxial test may result in different structure of the soil then obtained by field compaction by sheep foot roller. It is usual to reduce the design parameters on this account. The soil samples were prepared by dynamic or static compaction for the samples of Beas and Shalandi dams. Corresponding undisturbed samples were obtained from the embankments. The results of triaxial shear tests are compared in Fig.7. CH soils of Shalandi dam gave lower values of effective cohesion c' and higher values of effective angle of shearing resistance ø'. In case of SM soils of Beas field compaction showed a higher value of c' and lower value for ø'. Such observations will permit change in the design during the construction and eliminate need of conservatism on this account. It will increase degree of confidence in design.

5.0: PREDICATED AND FIELD SETTLEMENTS:

Importance of field observations of environment and thin stratifications is heigh-lighted by case of Jund-Kandla Rail embankment. It is resting on thick soft marine deposit alternated by thin sand deposits of floods. The top 3m. of marine clay has desicated by alternate cycles of tidal variations of flooding and drying. This layer is stiffer than lower layer(Desai '68) Theortically total settlement of 56 cm after 150 years was estimated. To reduce period sand drains were thought of. Writers suggestion of a test embankment with filter at ground level has indicated a maximum of 30 cm. settlement in a years. The trend of settlement do not indicate appreciable settlement thereafter. The field observations indicated that change in stress will not be as expected theoretically and effect of thin sand stratifications could not be accounted for theoratically. Thus soil Mechanics applied with judgement could eliminate sand drains etc.

ACKNOWLEDGEMENT:

Such investigations are impossible without co-operation of many research workers. Assistance of all and Shri. K.S.Khilanani and B.K.Saigal in particular are acknowledged hereby.

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PROC. 39th ANNUAL MEETING OF CBIP, SYMP. ON STANDERDIZATION
OF RIVER VALLEY PROJECTS

STANDARDISATION IN RIVER VALLEY PROJECTS WITH REFERENCE TO SOIL ENGINEERING PROBLEMS

BY

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AND

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INTRODUCTION

Soil Engineering has been a recognised field of Engineering which cannot be overlooked in problems of large amount of excavation and earthwork have been done in irrigation and power projects in the recent years, may it be for foundations, coffer dam or earthdams etc. The method of selecting best site having least engineering problems in first two plans have pointed to the necessity of clearer understanding of each site.

The data required for selected Foundation site was meagre such as crushing strength of rock in foundation and durability of stone for construction. The millions of cubic yards of earth which can never have homogeneous character (like a stone) necessiated collection of vast data for foundation and construction material. The subject being empirical and not scientific more data during pre-construction and post construction became obligatory. Dr. Terzaghi has rightly stated, ."Soilmechanics will not survive till practicing engineers come to realise that it is a supplement and not a substitute for common sense combined with knowlege digested by experience.

Collection of large volume of data during investigation construction and observation of behaviour has now been a normal feature for all the projects The larger the volume of data, the safer the design has been fashion of today. The records of all the construction methods, pore pressure pizometers, settlement gauges are planned at every site but its utility only depend upon its invelligent use. Br. Tezaghi's view, that percentage of useful information may range from 0 to 99% depending upon the qualification and experience of man who did the work does not seem to be any exaggeration. Even an excellent collection may require weeks of concentrated efforts to thoroughly digest and condense the data for use.

Thus standardisation of certain steps and methods of presentation of data is a must for current use as well for keeping the information handy for future where identical site conditions would be met with.

DEFINATION OF PROBLEMS

It is necessary to define clearly the nature of investigations to be carried out. The problem can broadly be of natural behaviour of soils under additional stresses and release of stresses, or behaviour of remoulded soils under different conditions. The clear defination of problem will also indicate all the limiting conditions such as availability of particular type of earthmoving machinery etc. The problem for construction materilal for instance can be very widely classified as under:

1) Qualitative and quantitative survey of types of available earth materials for embankment construction.
2) Quarry mapping and study of complete ranges of Engineering properties of selected borrow areas for design of dams.
3) Study of economics covering types of soils, loads, lifts and nature of machinery to select the zoning of dam.
4) Study of field behaviour of soil under test embankment.

EVOLVING INVESTIGATION PROGRAMME

On the basis of above defination and site inspection, outline of investigation programme can be broadly defined. The rest of the details will be evolved during the investigation. It is a chain process and is very difficult to standardize.

For foundation problem involving seepage studies undisturbed sampling, insitu testing can be planned at 200' and 100' interval in general. From the data if sand layer is observed as shown in fig. 1 on bank, it becomes obligatory to define its extent and its probable contact with river sand. In that case each hole drilled can only guide the programme of another.

In general the programme will cover all the possible questions that will arise to provide a constructive solution to problem. It is usual to find a project in which millions of cubic yard of compulsory excavations have not been considered for utilising as borrow materials for dam in the initial stages of design.

TESTING PROGRAMME

Large number of tests are required under wide range of specifications, which will never be identical for two projects. Even for identical projects the methods of design are unfortunately not standardized thereby resulting in wide range of test procedures within some research units. Consultants of different countries insist on different test procedures which do not permit of onforcement of available Indian standards.

The approach of simple statistics in averaging soil properties are most dangerous trends. The sampling of soils ~~for~~ and special shear test will have to be done on representative samples which will be selected by suitable process.

For borrow area material, bulk samples in few hundred are tested. They may indicate range of angle friction from 0° to 30°. The statistical average in any case do not represent the average properties of material in section of dam because the use is not in planned way. In such cases the field examination of borrow pits must be used as a guide to produce sketchy quarry map giving description of soil type available in each quarry. The quarry map will show the extent of soil in plan and will specify the depth of cut as well as the probable type of equipment for cut in vertical (showel etc.) or horizontal (scraper etc.). On the basis of thi;s 10 to 15 samples can be taken, for the specified cut by identical process to recommended construction method (individual sample or composite sample for scraper or showel cut), to demonstrate the boundary. Depending upon the area few samples can be taken by grid pattern and tested for classification tests. The data thus collected can be compiled in form of a graph as shown in Fig. 2 to show range of grading limits and Atterberg's limits for the material. Similarly for foundation, the identical soil profilo can be logged and classification tests can be used to plot the foundation material at each layer on figure similar to Fig. No. 2.

SELECTION OF SAMPLES FOR ENGINEERING PROPERTIES

A. Compaction Test:

Having covered the limits of grading of soil and Atterberg's limits, 2 samples of each typical extremes and average grading soil can be tested for the proctor dry density and optimum water content.

Thus fig. 3 can be plotted to show the range of compaction curve for the quarry. Thus a broad outline of maximum and minimum density with range of OMC can be determined. The question then is to decide the compaction machinery and field moisture for achieving it. In an area like Assam where natural water content is expected to be high above the optimum and most part of year rains are expected, Laboratory OMC has no meaning. Similarly better compacting equipments can achieve higher density at lower OMC. The design must take account of it from the preliminary stage.

On all the major projects it is worthwhile running test embankments to find economical compacting device by combinations of equipments and other perameters as water content thickness of layer, number of passes etc. The final study will then, give a range of possible limits of density and moisture content for construction stage. This will not only help in correct estimate of strength but also will help in evolving most economical specifications for compaction. The data collected will be available for use in other projects having identical conditions and materials.

The procedure of specifing density at 98% of proctor etc. is not normally advisable unless the uniformity of soils for the total requirements of dam is assured. This process have resulted into heterogenous density and moisture condition within the section due to different soils available for construction. The best way therefore is to specify range e.g. 1.68 to 1.7 g/cc. at placement water 14 to 16%. The placement water content has also to be decided in light of construction pore presssure and settlement.

B. <u>Consolidation Characteristic:</u>

The next step is to compact the soil in odeometer and consolidate it at placement water content to determine percentage consolidation. This can be used to estimate construction pore pressure by Hilf Method. The data obtained for average or most compressible soil sample under the specified range of densities and moisture are given in Fig. 4.

Samples under identical condition can then be tested after allowing full saturation to occur to estimate coefficient of compressibility and permeability in odeometer. These values will be useful in estimating total settlement of dam.

C. Permeability:

For the most of the soils it is possible to judge in order to classify the soil as impervious, pervious and semi-pervious. Tests at range of densities on compacted samples will provide accurate data for calculating seepage losses and design of filters etc. It can be done on average grading of soil sample.

D. Shear Properties:

This test needs to be done most carefully on the weakest sample. One or two samples of weakest type i.e. clayey and more plastic type and one or two having average grading can be selected.

All the samples will be tested at lowest density and highest moisture specified. The test can be done to represent placement or construction conditions, seepage condition or consolidated undrained conditions using identical range of the pressures inconfinement identical to proto type. The observations of pore pressure volume change during shear and nature of failure as in the Fig. 5. The effective stress Mohr circle or vector curves can be plotted then for the various conditions. The testing on 2 to 3 samples can be averaged for design value under each condition.

This method seems to be more reasonable than to have 12 or 15 different samples tested at different OMC - MDD. The average of this test can be misleading whereas such contingency will not arise in the above process. Also these limited tests on a few samples can be done more systematically and under different conditions.

REPORTING OF DATA

The complete data can be summarised on one drawing as shown in the Fig. 6. This can be standardised to evove uniform pattern of compliation and reporting. The data can be studied at a glance and it *gives complete picture of ranges.*

Similarly separate drawing for hearting soils, casing soils and foundation soils along with the figures showing the site plan and borrow areas as shown in Fig.1-6 will enermously simplify the design studies and will not leave anything to doubt.

CONCLUSIONS

1. *Standardisation of methods of reporting data will be very useful for the designers. It will serve as useful record for keeping data handy for similar problems in future.*
2. *The procedure of recommending the statistical average for study of behaviour of mass earth work is not at all suitable method but it has element of risk covered in it.*
3. *Large number of tests at different densities moisture conditions as usually been provided in project reports is not a good practice. The procedure of selecting samples seems to be more reasonable and sound.*
4. *The fruits of standardisation of course are not direct for this type of works but can reduce considerable amount of labour at various stages and various levels in understanding, digesting and interpreting the data.*

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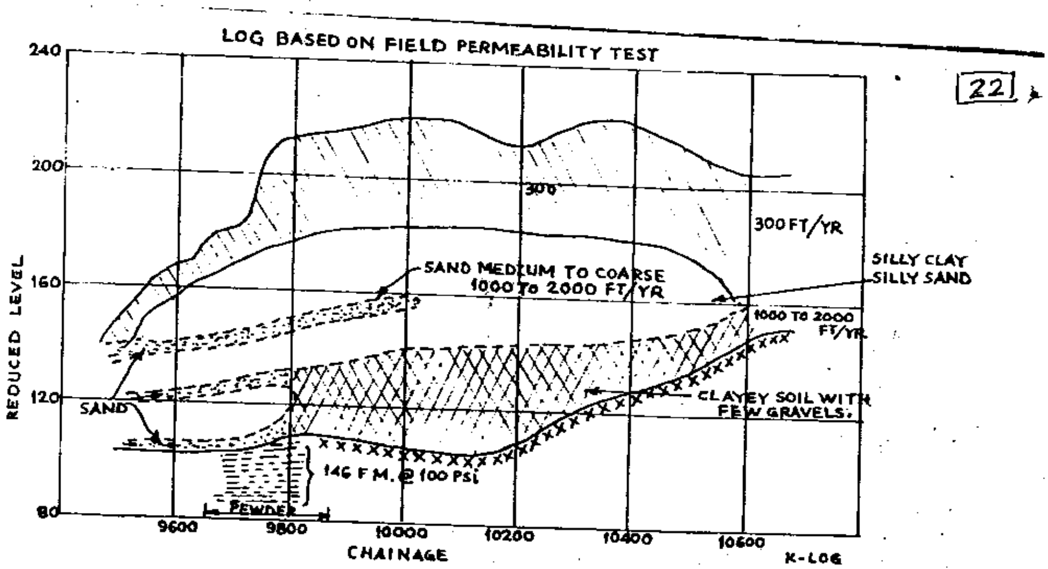
The authors are grateful to Shri C.L. Handa, Chairman, Control Water & Power Commission for inspiration and permission to publish the Paper. Kumari Champa Mansharamani, Research Assistance's assistance is also acknowledged.



SECTION OF SOIL PROFILE AT FOUNDATION OF EARTH DAM

FIG.1

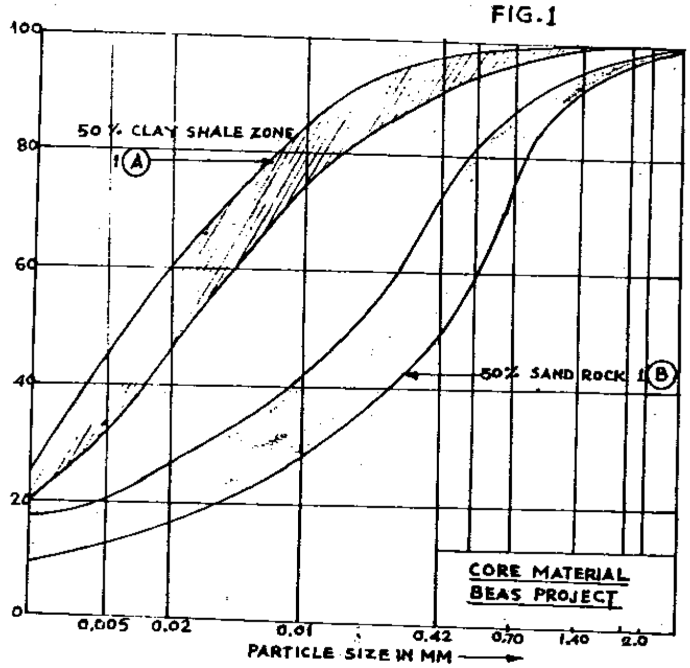


FIG.2(A) GRINDING CURVE

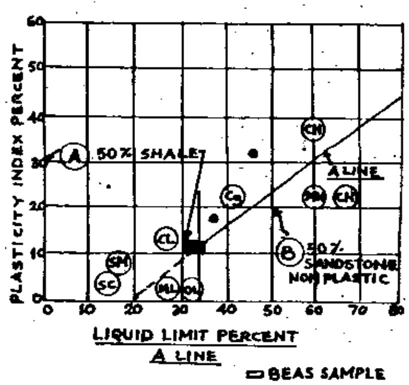


FIG.2(B)

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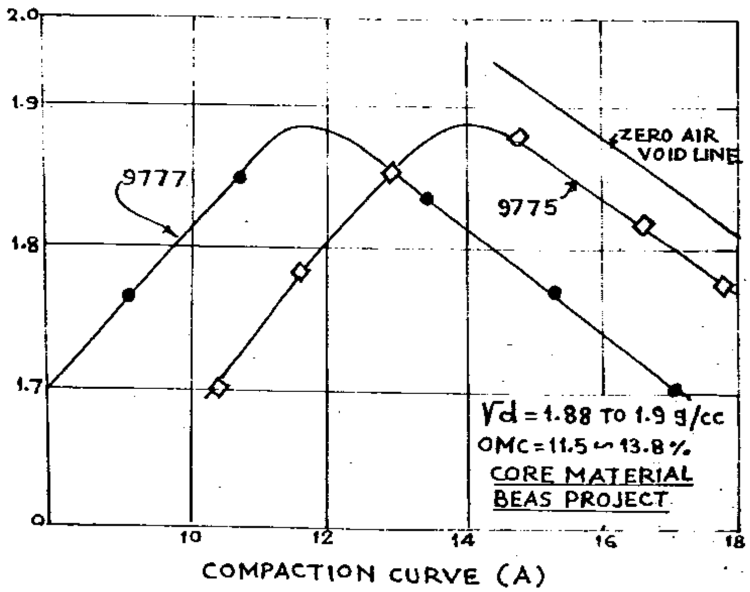


FIG. 3 (A)

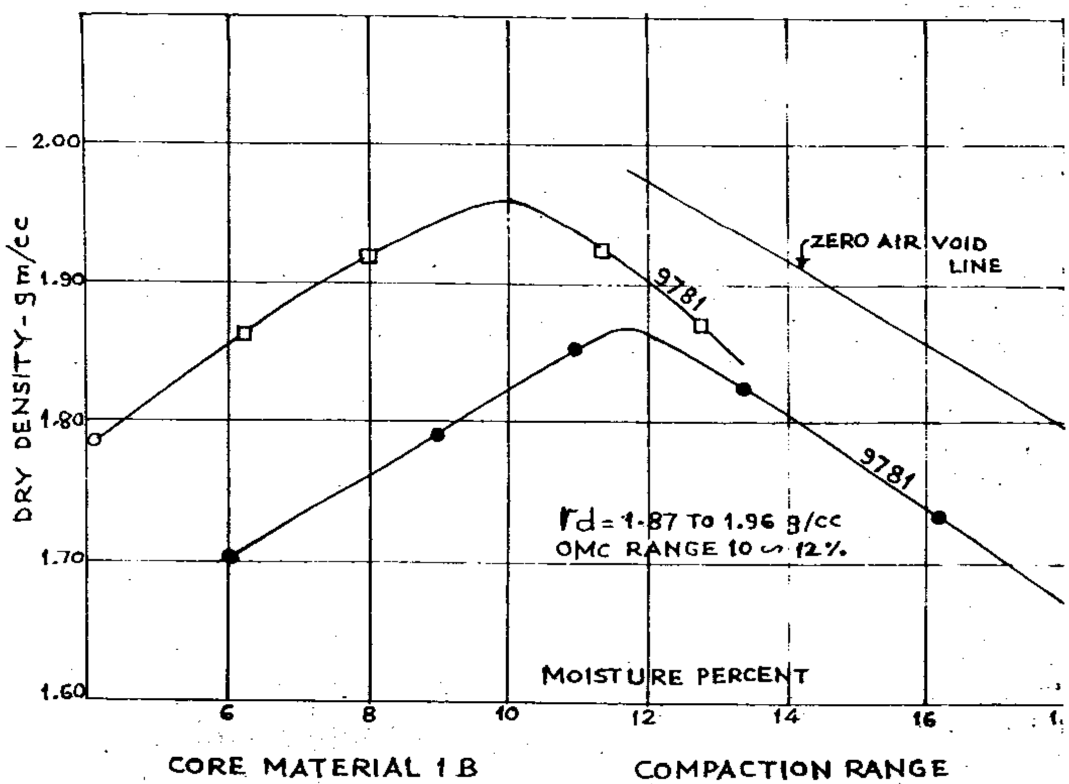


FIG. 3 (B)

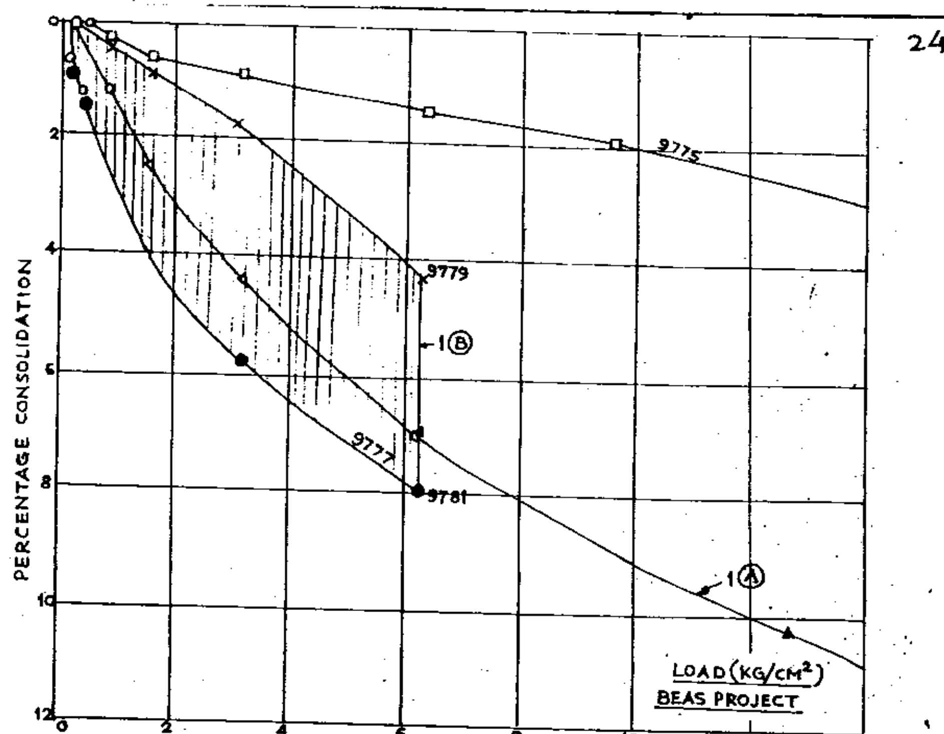


FIG.4(A) CONSOLIDATION TEST AT OMC.

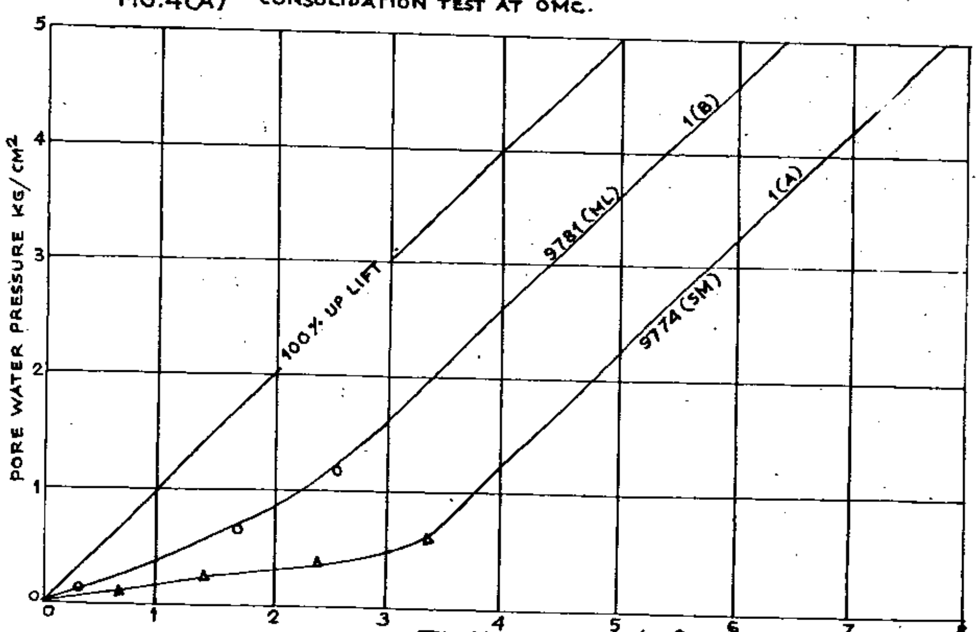
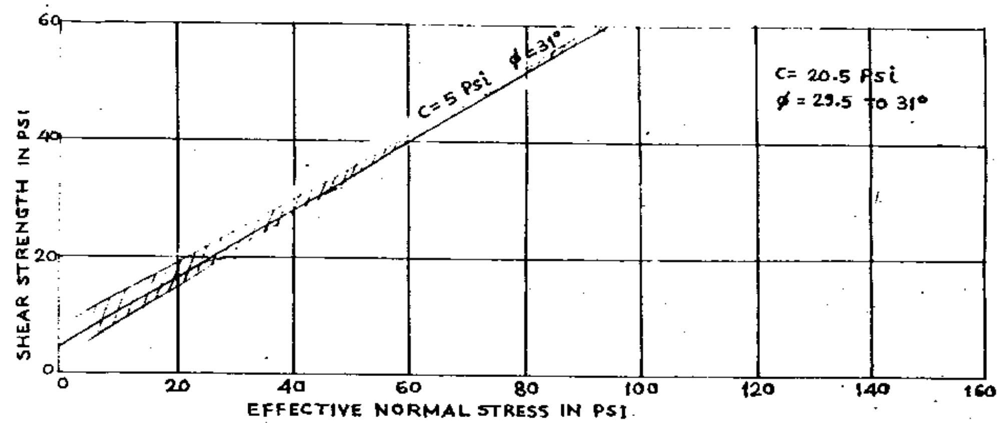
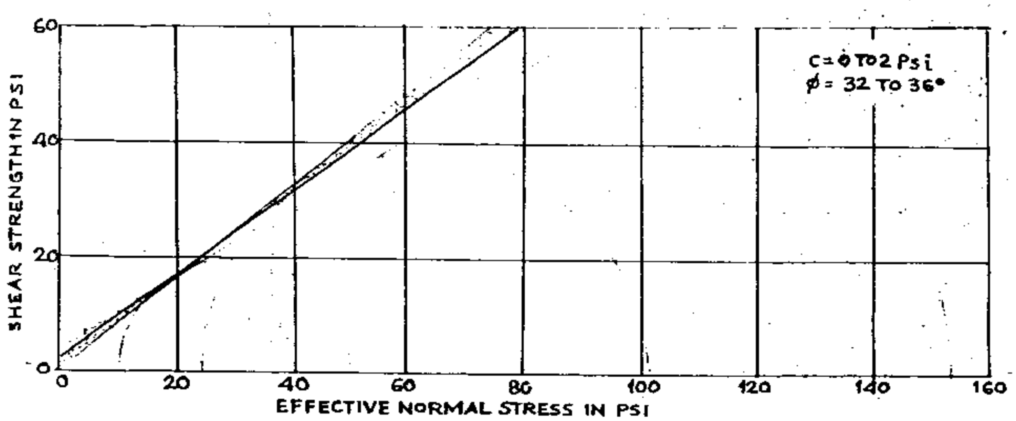


FIG.4(B) PORE PRESSURE CURVES (HILF METHOD)



SHEAR TEST AT OMC FOR BEAS 1 (A)



SHEAR TEST AT OMC FOR BEAS 1 (B)

FIG. 5(A)

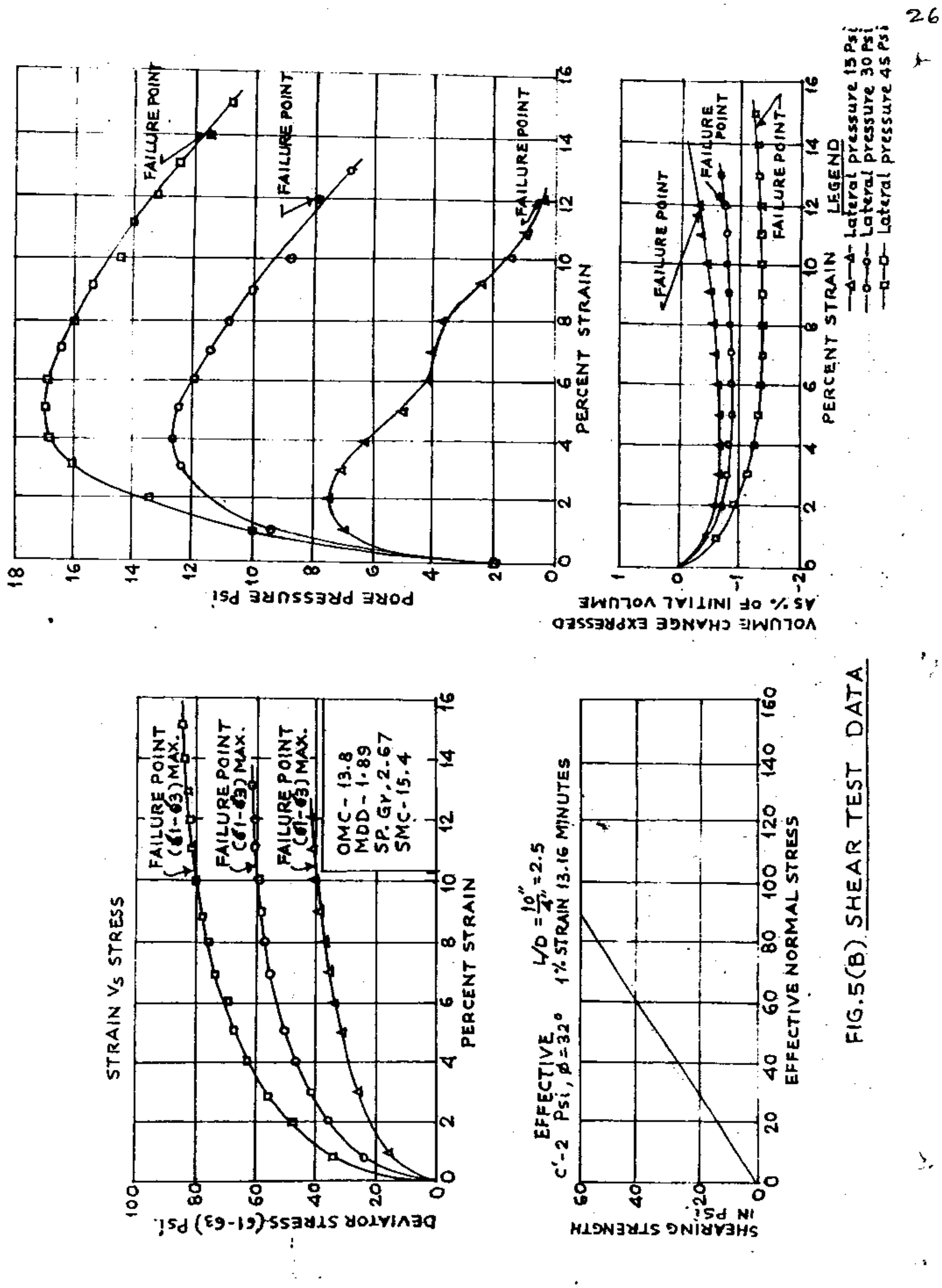


FIG.5(B) SHEAR TEST DATA

DRAWDOWN IN EARTH DAMS

M.D. DESAI* T.B. DESAI** D.D. PARMAR***

SYNOPSIS

The review of working of dams in India is used to bring out common characteristics such as flat cores and heavy total drawdowns. The condition of a rapid drawdown in such cases necessitates flatter upstream slope. The common concept of drawdown is discussed and modified to incorporate factors like time, porosity, water conductivity and upstream slope angle. Attempt has been made to provide a critical value of coefficient of permeability for a given slope made of earthfill having known porosity for a given rate of depletion of the reservoir. Data of behaviour of Ramganga dam is used to verify the range of the validity of results.

INTRODUCTION

Though earth dams older than 1000 years in India have been reported, large scale dam construction with empirical designs appeared to have started after 1870. Since independence this activity was accelerated with some scientific approach. During the plan periods, 31 earth dams with height varying from 20 to 120 metres have been planned and executed. Most of zoning and designs were still based on empirical approach with little scientific bias. The behaviour of dams built and experience of scientific approach in other countries have guided major conceptual modifications in the recent past. The design practice requires checking of upstream slope stability under the drawdown condition. The basic concept of drawdown and its effect on slope stability is critically discussed in the paper.

CHARACTERISTICS OF DAMS

Types of Earth Dams

About 65 per cent of the dams built in India are earth or rockfill dams. All of them are rollfill type. Hydraulic fill dams have not found favour so far. Very few dams like Kota, Sharavathy and Salal are rockfill type. Even homogeneous dams like Sharda and Tawa are negligible in numbers. Most of the dams are zoned with a core and a casing, latter usually is higher strength prism but not necessarily pervious. Inclined cores with chimeny filters have been recently introduced as in the case of Beas, Ukai and Ramganga.

Core Zone

The core of dams made of highly clayey impervious soils occupies major portion of dams built in the past. Thinning of cores, use of transitions and randomfill on the downstream of cores have been introduced recently. The hearting or core zoned occupied 40 to 70 per cent of total cross section in most of projects completed after 1948. The core is made from low shear resistance clayey soils having low coefficient of permeability (10^{-6} to 10^{-4} cm/sec) prohibited soils of SC-GC-SM-ML groups for core by convention in the past have now been used at Beas and Ramganga as core material. This was made possible due to advancement in soil engineering, quality control techniques and earthwork engineering. This high resistance soils in core led to introduction of thin inclined core recently for Salal, Kishau and Tehri dams.

Casing Zone

Casing or shoulder zone was nonexistance in earlier sections. It has been introduced as stronger

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supporting zones on both upstream and downstream. The zone invariably had more permeability than core (10^{-1} to 10^{-6} cm/sec) and is commonly termed as semi-pervious zone. Sandy and gravelly clays, loams and silty soils or disintegrated rock (locally known as Murrum) have been deployed. In general casing soils have been more heterogeneous. Conventional practice of avoiding sand and gravel and rockfill in shoulders has been broken with Beas, Ramganga and Salal. The casing zone varied from 30 to 85 per cent of the dam section.

DESIGN OF DAMS

The earth and rockfill dams having height varying from 20—124 m required 3 (Sharavathy) to 23 million cubic metre (Ukai) of total earthwork. Recent design of Beas (107 m) and Ramganga (132 m) contained 32 and 19 million cubic metres of earthfill. The upstream slope of conventional flat cores dams varied from 1 : 3 to 1 : 4 in most of the cases. In the case of Beas, Ramganga and Salal it varied from 1:2.2 to 1:2.5 (Yokovlev 1967).

Drawdown, conventionally defined as height of drop of reservoir from full reservoir level (FRL) to minimum reservoir level, varied from 30 to 80 per cent of the height of the dam. Such a high drawdown with nearly impervious casing zone on the upstream affected stability of the upstream slope. Need of reducing the effect on upstream slope was felt in the case of Tawa, Kota and Sharavathy projects. Some measures to reduce drawdown pressures like horizontal sand filters have been introduced in the projects like Tawa and Damanganga but it seems that the advantage of such measures to minimize the earthwork has not been availed.

DRAWDOWN

The commonly used definition for the drawdown i.e. drop of FRL to a minimum reservoir level is assumed to be instantaneous or rapid in the design of upstream slope of earth dams. The assumption implies that reservoir water level drops so fast that the water in voids of the dam has no time for draining. The soil mass in slip which had submerged unit weight will act as saturated. Also it will result in hydrostatic pressure and reversal of seepage pressures, thereby reducing the factor of safety under steady seepage state. In fact reduction is mostly due to development of high pizometric pressures. It is common to neglect change in pore volume during rapid drawdown, the pore pressures will be dominated by gravity seepage. In any case this pore pressures cannot be independent of the rate of drop in reservoir and soil permeability.

Thus concept of drawdown is an extremely conservative assumption. Physical probability of such rapid drawdown would mean wash out of major part of the dam to release millions of cubic metres of storage. Even for such remote critical design criteria usually specified factor of safety is 1.5 sounds to be illogical. In normal operation, the reservoir will drop probably from maximum to minimum in a year or two if there is no inflow. Then assuming year as unit of time for drop are we justified in taking drawdown of 6 m at Panset Hill, 10 m at Hirakund, 20 m at Tawa and Tenughat, 24 to 25 m at Sharavathy, Balliwella and Ukai and 43-45 m at Beas and Ramganga at par i.e., very rapid drawdown? It not only sounds illogical but also inexplainable scientifically. The drop of reservoir for a maximum draw of water from all outlets will vary as illustrated for Ukai capacity curve in Figure 1. For a fixed withdrawal of 50,000 hectare metres/day drop increases from 0.7 m to 5.2 m/day when reservoir level drop from FRL to minimum R.L. The rate of drop is a function of characteristics of reservoir (Elevation-capacity curves) and normal drawdown is very small at FRL. It is therefore desirable to consider rate of drop and not total drawdown in analysis.

LIMITING STATE OF TRANSIENT FLOW

For a drawdown against unconfined aquifer, the drop of free surface will be a function of porosity, rate of drawdown, permeability and geometry of upstream slope. Using Glovers (1964) work for a saturated bank subjected to cyclical empting and filling after initial transients have died out (Figure 2). Parmar (1977) derived the following expression for limiting case when the reservoir water surface is same as piezometric

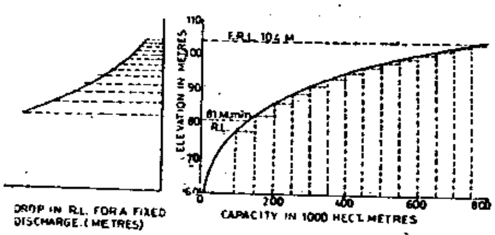

FIGURE 1 Elevation capacity curve showing level drop required for 50,000 hect. metres discharge/unit time

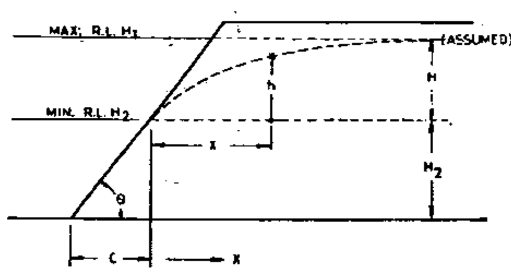

FIGURE 2 Limiting state of transient flow

free surface at $x > 0$.

$$\text{erf}\ \frac{x}{\sqrt{4\ \alpha\ t}} = 1 \qquad \text{...(1)}$$

where $\alpha = k/n' \times H$

The equation can be simplified as

$$k = \frac{n'\ x^2}{51.84\ h\ x\ t} \quad \text{where} \quad x = f(h, \theta) \qquad \text{...(2)}$$

Taking the upstream slope as phreatic line just after drawdown, θ = slope angle as shown in Figure 2. Thus the value of k so obtained will be that limiting value beyond which no residual piezometric head can be expected in the bank. Thus the drawdown will not create more adverse state than the steady seepage state of slope. The values of k for different slope angles θ (i.e. ½ : 1, 1 : 1, 2 : 1), rate of drawdown h/t (varying from 1 to 4 m/day) and porosity n′ (varying from 5 to 30 per cent) has been worked out and are shown in Table 1.

TABLE 1 Details showing permeability for different slopes, different rates of drawdown and different porosity

Slope of U/S θ	porosity n′ per cent	Permeability k for different rates of drawdown (in $\times 10^{-4}$ cm/sec) critical			
		(1 m/day)	(2 m/day)	(3 m/day)	(4 m/day)
½ : 1	5	4.50	8.90	13.40	17.86
	10	8.90	17.86	26.79	35.72
	15	13.40	26.79	40.19	53.58
	20	17.86	35.72	53.58	71.45
	25	22.33	44.65	66.98	89.31
	30	26.79	53.58	80.38	107.17
1 : 1	5	1.12	2.23	3.35	4.47
	10	2.23	4.46	6.70	8.93
	15	3.35	6.70	10.05	13.40
	20	4.47	8.93	13.40	17.86
	25	5.58	11.16	16.75	22.33
	30	6.70	13.36	20.04	26.72
2 : 1	5	0.28	0.56	0.84	1.12
	10	0.56	1.12	1.67	2.23
	15	0.84	1.67	2.51	3.35
	20	1.12	2.23	3.35	4.47
	25	1.40	2.80	4.19	5.58
	30	1.67	3.35	5.02	6.70

CONCEPT OF RATE OF DRAWDOWN

Using the above analysis, for a given slope having known porosity, coefficient of permeability a safe rate of drawdown for no piezometric pressure condition can be worked out. The actual rate of drawdown for a given reservoir characteristics can be worked out as shown in Figure 1. Comparing the two, the sensitivity of slope to drawdown condition can be assessed.

FIELD VERIFICATION

Data available on observations of Ramganga dam (Manglik *et al* 1977) have been used for an approximate verification of a theoretical concept evolved here. The slope of Zone II *i.e.* crushed rock zone is 0.5:1. The reservoir level dropped rapidly at a rate of 0.3 m/day during 1975. The porosity was about 30 per cent and k was 2×10^{-5} cm/sec. The k critical for zero piezometric head from the Table 1 will be around 6×10^{-4} cm/sec. It can be seen from figure 6 of reference (Manglik *et al* 1977) that only small residual piezometric head remained. The trend confirmed the concept of rate of drawdown though results may need more refinement and further work to reduce the gap between theory and practice.

CONCLUSION

The concept of rapid drawdown in design of upstream slope of earth dam is discussed and was found to be too conservative, illogical and unscientific.

A new concept to introduce rate of drawdown has been suggested. With the help of a theoretical derivation, correlation of k, slope angle θ, porosity n' and a critical rate of drawdown for a upstream slope can be estimated. If actual rate of drawdown is less with adequate factor of safety drawdown condition will not be critical for stability of slope.

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Discussion on Evolution of Design during Construction of River Valley Projects

Amar Singh (*Member*)

The discusser congratulates the authors for bringing forth the evolution of designs in a very nice way.

In 'Introduction', the authors have mentioned that: 'civil engineering is not an exact science'. The discusser thinks that it is partly true. The problems in civil engineering can be broadly classified in following categories: (i) problems which can be mathematically defined; (ii) problems which cannot be mathematically defined but can be solved mathematically by making suitable assumptions with good judgement; and (iii) problems which cannot be mathematically defined at all even with desirable assumptions.

The authors' observation is applicable to problems falling under category (iii) only; and not to problems falling under categories (i) and (ii). Dr Dhillon's observations are, of course, applicable to categories (i) and (ii). The problems falling under categories (ii) and (iii) can be given a scientific touch, if suitable experiments are conducted; and results so obtained, are applied with correction factors.

The authors also stated in 'Introduction' that: 'It is generally felt that once the designs have been prepared, at the inital stage after project investigations, they should be final and any change in the design at a later stage or during construction is probably due to lack of original concept of design.' This concept will prove a big asset, if the designer aims at it while working out his designs. It is experienced that change, if needed because of field conditions, will not affect the designs to a great extent in most of the cases. It is, however, admitted that the possibility of a big change cannot be ruled out in rare and exceptional cases. But, if the designer frames an impression in his mind that the design has to change at later stage, then he will not exert fully in his initial designs an undesirable attitude.

In case of Nagarjunasagar Dam, the authors have nicely described the failure of ribs in tunnel. The discusser would point out that it is a practice with the Bhakra and Beas Designs Organization to classify the geological strata along centre line of the tunnel as 'very good', 'good' and 'poor' reaches, because the strata which are always met in all cases vary in this order. The designer then produces typical sections for each strata accordingly, and these sections are incorporated in field in consultation with the geologist at the project.

Further, the discusser, while studying Fig 3, observe that penstocks embedded in the dam are cheaper (stated by the authors also) and safer proposition in view of damages which are likely to be done to penstock by possible cavitation. The monolithic behaviour between blockout and masonary, may be achieved by provision of shear connectors etc.

The discusser takes an opportunity to present some salient features of a lecture 'The Assessment of Load Factors' delivered by him. In this lecture, the discusser had mentioned that 'The designer's best assessment will be obtained from information based on a careful statistical study over a large number of years; his worst assessment will be when he has to guess.' The second important point in the lecture related to the main factors which influenced the possibility of collapse as enunciated by 'The Committee of Institution of Structural Engineers'. The discusser experienced that the designs do stand nicely in most cases, if the aforesaid points are kept in view by the designer.

The discusser would like to further present that a confusion is made between 'correct assessment of load' and 'correct analysis of structure' by some designers. These two aspects of detailed designs should be separately dealt with; and the results so obtained, should be considered by a designer with sufficient experience.

O P Datta

The authors should be congratulated for the exhaustive information contained in the paper. The following particulars regarding the Beas-Sutlej Link Project are mentioned in this connection:

Dehar Steel Penstock

Medium strength steel, conforming to ASTM-A537/67a Grade 'A' has been used in the fabrication of surface penstocks and Wye. The physical properties of steel are given in Table 1.

TABLE 1 PHYSICAL PROPERTIES OF STEEL

Type of Steel	Ultimate Tensile Strength, kg/cm^2	Yield Point Strength, kg/cm^2
ASTM-A-537/67a		
Thickness <31.75 mm	4 920	3 520
Thickness >31.75 mm	4 570	3 230

For the design of penstocks of Dehar Power Plant on Beas Sutlej Link Project, ASME Boiler and Pressure Vessel Code (Section VIII) has generally been followed. According to earlier 1965 edition of the code, the allowable design stress was based on factor of safety of 4 on minimum ultimate strength of steel for all categories of steel plates. In 1968, the ASME Code (Section VIII) was split into two sections namely Divisions 1 and 2. As per practice followed by USBR, the straight penstock sections are designed according to Division 2 of the above code which allows a factor of safety of 3 over the minimum ultimate strength of steel. This is not only applicable for high strength steels but for mild steels also. The complicated structures like bifurcations (wyes) are designed according to Division 1 of ASME Code which allows a factor of safety of 4 over the ultimate strength of steel. The same criterion was adopted for the design of penstocks of Dehar Power Plant.

Dehar Power Plant

Although not mentioned in the paper, the earlier project report of Dehar Power Plant envisaged installation of six units of 106 MW each. However, with rapid growth of load and likelihood of additional thermal units coming up in the region, the possibility of increasing the size of the units was explored. Ultimately it has been decided to install in all six units of 165 MW each. This increase in installed capacity was achieved without any substantial increase in the cost of civil engineering structures.

Dr M D Desai (*Member*)

The authors must be congratuated to evolve clear hypothesis suggesting concurrent design during construction as obligatory for fast developing country. This, against orthodox 'no change' attitude is a bold attempt which is undoubtedly excellent one, provided all the relevent changes are brought out to make the new approach a success. Frequent changes in original design are often attributed to lack of initial design concepts and inadequate explorations or ideas of site conditions. These are not totally baseless. There are a number of cases to illustrate. In fact, every project will have certain fixed features settled after detailed explorations and certain features, will be kept flexible for developing on the basis of experience of actual construction. If axis of dam or height of dam is to be altered during some stage of construction it has to be attributed to lack of adequate initial concept and minimum data.

The authors experience that however exhaustive be the investigations, the designs for the projects have to be modified during the construction is a fact; but the reasons: namely, to suit site conditions; and to achieve economy and accelerate construction programme are not equally convincing. Let us not hide basic fact that most of our projects are undertaken on designs based on preliminary explorations or even occasionally without any explorations. Such frequent major changes in layout and designs have not only resulted in loss of time but of money and development at Barauni Refinary, Heavy Electricals, Bhopal, Heavy Engineering Corporation, Ranchi, Namrup Fertilizer Factory, a navigation lock and many river valley projects for the want of investigations of site conditions in time. This list can be enlarged and mostly the trouble in irrigation and power projects can be ultimately attributed to omission of the stage of detailed investigations or proper compilation and utilization of the exploration data. Even Dr Rao[2] agreed that investigations are not systematic.

The aspect of changes to suit site conditions is a best illustration of our method of analysis. Many designs are prepared by those who are niether conversent with the site nor have developed critical points to be analyzed during site inspections. The provision of high retaining walls to protect abutments of rock and provision of negligible dewatering for sandy river bed are example of lack of oneness of designer to the site. Foundations are designed as raft and pile supported rafts on basis of assumptions that safe bearing capacity is 0.5 kg/cm^2. During construction, the construction engineer feels that the assumption needs to be checked for safety. But the soil engineer finds the safe bearing capacity more than 2.5 kg/cm^2. Can such changes be considered as changes to suit site conditions

Changes for economy again are found to be mostly based on short term comparison of alternatives of a component of the project, for example, cutoff for a earth dam ignoring many factors affecting the long term economy of the project as a whole. Occasionally, the chain reactions of changes in component on other part of the project are missed completely. A project envisaged in 1960 finds that it is economical and obligatory to provide canal lining in one canal as late as 1971 when canal construction was undertaken. The basis is again assumptions like weed growth; loss of command and waterlogging, which seems to have been written from a book rather than experience. Could ten years' time not provide data of the permeability, soil characteristics and the observation of groundwater table or waterlogging in similar canal system nearby? Concrete lining is specified without any cost comparison with experience of other types of lining used for various projects in India. The area has mostly expansive soil deposits and lining, as provided, will cause such problems which may be worse than expected otherwise. Also economy cannot ignore cost of resurveying, relignment, redesign of structures and interest on capital cost over years till canal starts utilizing reservoir storage. Often the excuse of economy is used for making changes to cover initial deficiencies etc. In present stage of development with unpredictable availability of engineering goods and machinery, economical analysis is rarely justified. There may be water in reservoir but no turbines (planning since 1965) to avoid power crisis. There are projects where water is not available in reservoirs and hence there is a power crisis and condition of draught. Also modified design on the grounds of economy on present rates becomes almost otherwise by the time the revisions are technically approved, administratively sanctioned and budget provisions are made available. The number of stages involved are bound to take time.

Necessity of publishing the experience has been rightly emphasized by Dr Rao[2]. The meager literature inspite of Rs 12 000 crores inventment on projects may be due to contradictions of data and design philosophies and continuity of the persons in specific fields. There may be good decisions but no data are available to support. Also, there may be sufficient data available but the analysis will not justify the proposal.

The current techniques of project design can be classified as: (i) rigid based on worst expected conditions, and (ii) flexible based on average anticipated conditions or combination of (a) and (b). The proposed method of authors evolving design during construction is similar to 'learn as you go' or 'experimental' design introduced by Terzaghi in 1945. In any design of earth structures engineering, physical constants representing average anticipated behaviour with variable time need aproximation and high conservatism. Inspite of refined methods of analysis, ultimately an empirical design based on average experience is used. This 'average', widely used all over, led to major disasters due to a minor geological fault or factor like effect of saturation. This method is wasteful, not at all fool-proof and does not generate useful experience.

In design, as we proceed, actual observations and experience during the construction of same project are u to produce economical designs scientifically. The process involves defining of design parameters, anticipating behaviour of structure on the basis of average properties and providing field instruments to continuously compare the anticipated and actual behaviour of certain factors governing the design. These observations permit constrant changes in design to take maximum advantage.

This observational technique and the proposed authors' method may successfully be employed only under certain circumstances. There are certain limitations of our existing system and set-up for making the observational method a sucess.

The current design method do not generally indicate critical parameters and their estimated values are assumed in the design. Design of earth dam showing expected settlements and pore pressures at various stages will be an exception.

If design is to be based on observations, the technique, personnel and processing will have to be selected with great care. In general, the work is left to very junior technical assistants who hardly understand the im-

..ication of observations. Inspite of large-scale instrumentation in all river valley projects, it may be difficult to find a team of expert workers devoted to this field.

From stage of design to analysis of observations, existing set-up of frequent change in staff, leaving observations to semi-skilled workers as a routine, and analyzing data by officers not conversant with field and that too after long periods have elapsed, is bound to dig a graveyard for the proposed technique. The reliability of the observations, therefore, is a function of personal interest and hence unless expert cell specialized in this field is organized, observational design technique has more disadvantages. It is usual to find a helper pumping water at pressure more than expected pore pressure at tip for dearing piezometers. It is difficult to imagine the fate of observational technique if present set-up is not overhauled. Though attempt was made to make a central cell, it has not brought out much impact on the projects executed by the States.

This technique is not suitable for problems involving sudden collapse. Such components of projects will have to be carefully isolated and designed by rigid design technique.

In contractual works, any major deviations in designs, ..il benifit the contractor even though the cost of the work may be reduced. Also, on major projects deviations in parts, when all items of planning are in short supply, will upset the time schedule, cost and many occassions prove penny-wise pound-foolish. In a project, shear zone was met with at a depth more than the head of stored water. The zone was found impervious. In order to avoid its treatment and analysis, the site was shifted upstream. This resulted in loss of all previous explorations of borrow areas and foundations. During construction, tight schedule of diversion, necessitated quick decisions, positive cutoff with two diaphragm walls to reduce dewatering were excuted. The exploration latter showed that shear stringers do exit along new site. Test grouting was then attempted to confirm that it is almost impermeable.

Observational method has to provide for slow down of work in the event of unusual behaviour of structure initially. Also, the designers will have to be prepared for worst eventualities. In present set-up, designers are divorced from the projects as soon as construction starts. ..e field staff are competent to make minor changes ..hich may actually violate the basic philosophy of designer.

There is no doubt that full advantage of this method cannot be realized unless the engineer is throughly conversant with problem, makes continuous alterations of designs and procedures as the observations are obtained and has authority to act quickly upon his own decisions. Above requirements would practically mean placing direction of project in hands of one or two individuals. Terzaghi's success was due to authority and ability to take the responsibility.

The cases cited by the authors also do not actually fall in category of plan as you construct from the inceptions of project.

Therefore, it is felt that before venturing on to apply the experience, a vigorous attempt may be made to strengthen most neglected research cells. Even if 0.2% of the cost of projects would have been invested and conducive atmosphere provided for research, the country would have today seen technological revolution.

How far the existing set-up can be overhauled? Can changing coordinator keep an up to date knowledge to coordinate specialized construction activities on major projects. Particularly when the present system is based on dividing responsibilities and localizing credits?

Flexibility cannot be adopted in design midway. It has to be planned from the begining. Earthmoving machinery, for example, has limited availability in our country and designer's abrupt change will have to be rejected outright for want of suitable machinery. Diversion scheme in a project required excavation of millions of cubic metres of earth. The soils in depth of 20-30 m cut were not for use in dam initially, when diversion was planned. Later on its compactibility posed problems which necessitated use of pneumatic rollers. As the work could not wait for importing these, the designer has to accommodate with available materials.

For various reasons, inspite many completed projects, experience of construction is limited to few and mainly to those belonging to contracting firms. The strain of satisfying the needs of audit and accounts is more on the engineers than the strain in executing technical jobs systematically. Technicals mistake can be covered but the failure of accounting and red tapism may ruin career of an engineer. This phobia cuts the very root of the new design method.

Now technological revolution will be like green revolution. Research requires devotion and devotion cannot be expected from all employed engineers. Thus it is not too late even now to start building up of research stations with proper personnel having aptitude and devotion to field of applied research.

With this base, above technique can at least be used with full benefits for same parts of projects in Fifth or Sixth Five-Year Plan.

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Prof M G Jog (*Member*)

The discusser congratulates the authors for producing such an excellent paper. However, he would like to offer the following comments.

Nagarjunasagar Project

When a single unit of 60 MW was proposed what was the change in the specific speed of the unit? What was the effect of this change on the setting of the turbine to avoid cavitation? How were the draft tube dimensions modified?

When the power plant was modified to serve as a pumped storage plant, the machines must have been changed. What was the effect on the power station layout?

Sabrigiri Project

What is the centre line of the runner?

If the columns had been of RCC, what would have been the additional expenditure?

Lower Sileru Project

Would it not have been safer to locate the valves in a separate cavern though the cost might be increased?

If the shaft can be so lifted that its topmost point is in level with the topmost point of the crane trolley, could the height of the power house be reduced?

Would providing for downward dismantling of the runner reduce ultimately the overall costs?

How would the draft tube gates be operated?

Y K Murthy, K Madhavan and (Km) E Divatia (*Authors*)

The authors are very thankful to all the discussers for taking keen interest in the paper and raising certain queries. The replies to their queries are as given below:

Shri Amar Singh has made observation that the statement made by the authors that : 'civil engineering is not an exact science' is partly true and he classified the design problem into three categores. There are many varying parameters to be considered in civil engineering design and mathematical solution of any type of problem. Suitable assumptions for various parameter within permissible limits are required to be made.

He has also stated that the designer may not put his best initially if he has an impression that the design has to change at a later stage. However, the authors feel that no designer would like to go on changing his designs unless essential.

He has also pointed out that the practice with the Bhakra Beas Design Organization is to classify the geological strata along central line of tunnel as 'very good' 'good' and 'poor' reaches. This type of classifications is hypothetical and will not help to assess the load exactly. In fact, it is well recognized by tunnel designers who have studied tunnels under different geological conditions that classifications of rock loads on the basis of some well known classifications like that of Terzaghi and Protojadikonon are liable to great variations.

The point mentioned by Shri Amar Singh about cavitation is not clear since the penstocks are usually designed under positive pressures. As indicated by him the design load factor is finally fixed on the basis of the probability and this concept has already appeared in most of the design codes in the world.

Shri Datta has amplified the design criteria adopted for the penstocks of Beams Sutlej Link Project.

Dr Desai has emphasized the need for field investigations before projects are undertaken for construction. He also feels that there should be good coordination between the design and research in order to obtain the maximum benefit from observations of the behaviour of the structure. Research organizations should be strengthened and manned by specialists in the fields. He has also emphasized in general terms the need for a coordinated thinking in the planning, investigations, designs and construction of major river valley projects. He expects that all the aspects enumerated above could be beneficially utilized at least in the Fifth and Sixth Five-Year Plans.

The authors agree with the views of Dr Desai as these are the essential aspects for executing major projects.

The authors are thankful to Prof Jog for taking keen interest in the paper and raising the queries. The various points raised by him are replied as below :

Nagarjunasagar Project

The specific speed of turbine was kept same and it was approximately equal to $\frac{632}{\sqrt{H_d}}$. Since specific speed is same, the setting level was also not changed.

The draft tube opening was changed from 2 of 7.62 m to 3 of 7.925 m (Fig 1).

The machines for pumped storage scheme are reversible pump turbines and to reduce cavitation, while operating as pumps, these require lower setting. The change in the layout is still under study in consultation with manufacturers.

Sabrigiri Project

The centre line of the runner is 195.372 m.

Steel superstructure was already constructed. The instability of the steel frame superstructure was observed during EOT crane operation. The question of comparing the cost with RCC structure did not arise.

Lower Sileru Project

The present practice is to locate the valves in the machine hall itself irrespective of an underground power house or surface power house. Lower Sileru being a semi-underground power house, for the pressure shaft a minimum rock cover will have to be ensured. Further speparate cavern for valves would involve installation of a separate heavy crane.

The crane beam elevation is fixed for untanking of transformers in the erection bay at an elevation of 94.000. The service bay is located at an elevation of 86.000. The conditions as mentioned by the discusser was not critical in this case.

The downward dismantling of the runner does not effect the overall cost. However, it facilitates the removal of runner for inspection and repairs without disturbing the generator and rotor alignment.

The draft tube gates will be operated from draft tube deck at an elevation of 94.000 by gantry crane which is not shown in Fig 8.

Paper by Y K Murthy, *et al*, *Journal*, vol 53, pt CI 3, January 1973, p 113.

Proceedings of the Conference on Construction Practices and Instrumentation in Geotechnical Engineering — Surat/India/20-23 Dec. '82

ANALYSIS OF DISPLACEMENTS WITHIN EMBANKMENTS FOR DESIGN OF INSTRUMENTATION

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1 INTRODUCTION

Though methods for computing average factor of safety of embankments have been fairly standardised, it can be visualized that the soil in a portion of the slip plane may have been stressed beyond its strength. This will initiate progressive failure. In addition to the slope stability, the stress displacement study will be required to complete the analysis.

An earth embankment is heterogeneous anisotropic, finite mass having soil in any state from elastic to plastic state of equilibrium. To initiate the stress analysis, deformations which control the mobilized shear resistance as well, will provide an indirect key.

2 DEFORMATIONS IN EMBANKMENTS

Any soil element in an embankment would be subjected to three deformations vertical, upstream - down - stream and axial. The analysis of these displacements alone could give an idea of the damage to core, outlet works and embedded structures and progressive failure. The earth pressures on embedded retaining structures cannot be correctly evaluated without the knowledge of displacements.

Theoretically, predicting deformations in the embankments with wide range of the soil along length, width and height, compacted by different plants at variable placement OMC, will not be easy unless an idealized model is evolved. Thus an indirect approach of predicting the deformations based on the performance of well instrumented structures has been attempted here to initiate the process to evolve a model.

The increasing cost of earthwork in embankments is bound to influence the designer to accept "Design as you observe" or "Casagrande's calculated risk" approaches. In both the cases measurements of the important design parameters is a must. This in turn requires prediction of the probable range to select instruments having desirable accuracy.

Not only the proper selection of the instruments having maximum sensitivity but also applications of the modern tools of analysis such as finite elements will depend on the availability of models which permits prediction of deformations, pore pressures, etc.

3 SCOPE OF WORK

The scope, considering the introduction has been limited to evolve an empirical model to predict deformations in embankments based on the performance of well instrumented dams of the world. The details of deformations recorded by cross arms, inclinometers, slope indicators and surface gauges for five dams have been compiled from the published reports and papers. All important details of the embankments have been summarised in Table 1.

4 VERTICAL DISPLACEMENT

The vertical movement recorded at different heights (above the base) are analysed for (a) Construction stage (Sc) and (b) total settlement (St) for five dams. Percent vertical deformation is expressed by

$$\lambda\% = \frac{S}{(H - h)} \qquad \dots\dots\dots (1)$$

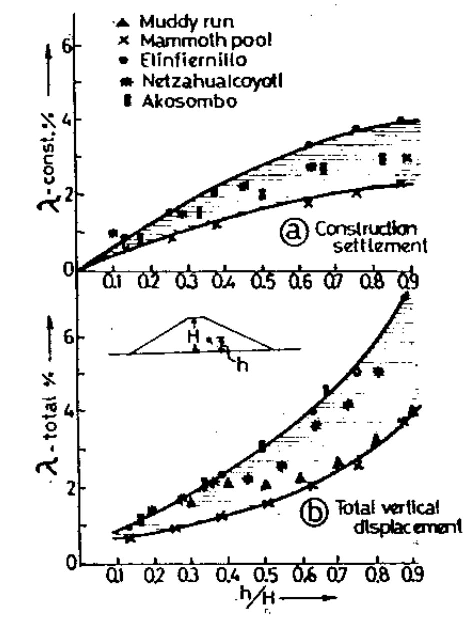

Figure – 1 Variation of vertical displacement factor λ with height (a) Construction (b) Total.

λ, λ_c = % Vertical deformation total and during construction

H = Total height of dam

S = Vertical deformation in cm.

The observation of λc and λt_{otal} have been plotted against h/H for above projects in figure 1 (a) and 1 (b) respectively. The variation of the total vertical deformation; even for heterogeneous conditions, exhibit a regular pattern as shown in Figure – 2, the maximum St being shown at almost mid-height of the dam.

The correlation of the results have been statistically worked out for upper and lower boundaries as given below :

$$\lambda t = 0.00376\, n^6 - 0.08588\, n^5 + 0.772\, n^4 - 3.4606\, n^3 + 8.124\, n^2 - 8.8635 n + 4.83 \quad \ldots\ldots\ldots (2)$$

λt (Lower bound)

$$= -0.00018\, n^6 + 0.00629\, n^5 - 0.0745\, n^4 + 0.411\, n^3 - 1.11\, n^2 + 1.7678 n - 0.24 \quad \ldots\ldots\ldots (3)$$

where $n = 8 \times r_p$, $r_p = h/H$

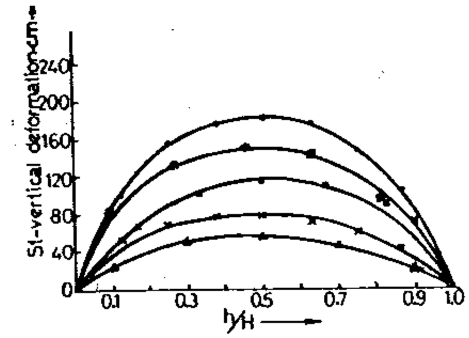

Figure – 2 Pattern of variation of St total vertical displacement with h/H

To verify the applicability of equation (2) and (3), case records of Beas dam (Verma, R. K. 1978) are used to compute maximum, minimum settlements for different height h (H = 110 m) above the base. The field observations of performance compared with computed values in Figure – 3 shows that above equations are fairly reasonable in predicting the value and trend of vertical deformation in embankments.

5 MAXIMUM VERTICAL SETTLEMENT

The results of analysis for the five embankment dams gave following expression for estimating total vertical settlement for the core :

$$St = 0.053 \times h \quad \ldots\ldots\ldots (4)$$

The height of embankment has to be fixed taking into account settlement St such that even when it has come to equilibrium, the design free board remains unaffected. The common practice is to use Gould's (1954) chart for 40 m high fills. There are no other recommendations for higher embankments.

If the modified equation (4) St = 0.06 h is adopted, the Table – 2 below will justify its use with adequate safety.

The recommended value by Gould for the above embankments are 40 to 60% higher than actually observed total vertical settlements. The results have also indicated that the maximum vertical deformation occurred around 0.44 h from the base.

Table - 1. Details of embankment dams

Dams	Mannoth pool	EL-infiernillo	NetZahual-Coyoh	Muddy-run	Askosmobo
Height (metres)	98.50	148.0	137.5	76.2	112.7
Length (metres)	250	344	480	1463	670
Type	Zoned Rockfill	Zoned thin core rockfill	Rockfill	Earth and rockfill	Rockfill
Construction period	1958-59	1962-63	1963-64	1966	1961-65
Core material	Fine silty sand	Highly plastic soil	Latteritic soil	Clayey silt	Highly Plastic clay
Foundation	Alluvial	Rock	Rock	Sloping rock u/s to d/s	Rock steep valley
Instruments embedded	Slope indicator, USBR cross arm, Surface monuments	Vertical Cross arm, Inclinometer, Horizontal extensometer, Surface monuments	Inclinometer, Surface monuments	Alignment markers, Inclinometer	Slopes Inclinometer

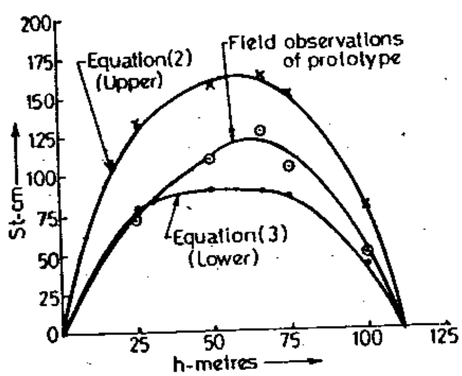

Figure - 3. Comparision of computed upper, lower bound values of St with actual observations of Beas dam.

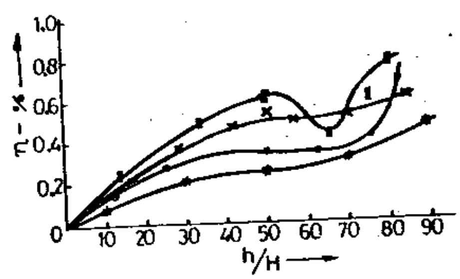

Figure - 4. Variation of co-efficient η for axial deformation with h/H

Table - 2. Total vertical deformation of some dams.

Dam	Height (H)M	St_c computed	St_o observed	$\frac{St_c}{St_o}$
Beas(Punjab)	110	6.0M	4.58M	1.31
Ramganga (U.P)	67	4.0M	3.86M	1.03
A dam in USSR (Vutsel '78)	150	9.0M	5.90	1.52

6 AXIAL DEFORMATION

The axial deformation or cross valley displacements were analysed and have been converted into a factor η as under:

$$\eta\% = \frac{S}{H - h} \quad \text{where } S \text{ is in cm.}$$

The results are plotted in figure - 4. In some cases displacements in two sections across the axis showed wide variation at some elevation. This is bound to create different state of stress in some parts of the embankments.

The variation of the axial movement along h exhibited symmetrical parabolic trend with zero displacements at top, bottom and maximum around mid-height.

The design of junctions of retaining blocks and earth dam will have to account for cross valley movements of variable magnitude at various heights. The forces generated will have to be accounted for three dimensional slip circle analysis. For all practical purposes $\eta\% = 0.025 \times h$ would be recommended conservative assumption.

7 HORIZONTAL DEFORMATIONS

The horizontal movement i.e. upstream downstream displacements, during construction are small at the top and the base. During filling of reservoir and thereafter horizontal movements have been noted in top 2/3 portion of the embankment. The maximum ultimatemovement has been observed at mid-height of the section. Using identical notations factor $\xi\% = \frac{S \text{ horizontal}}{(H - h)}$ has been evaluated and analysed along height. The variation for core and casing is highly erratic and covers very wide range. Maximum displacement ranged from 50 mm to 500 mm for embankment heights 60 to 90 m. The upper values of $\xi\%$ varies from 1.5 for core to 3.0 for gravel and rockfill.

8 CONCLUSIONS

Approximate empirical method, based on performance of five dams, have provided basis of estimating maximum vertical and axial displacements and its depthwise variations. The results of upstream downstream displacement analysis indicate trend of variation along height but the maximum value varies considerably with the rate of filling of reservoir, the casing material, etc.

The work has to be continuous evaluation to evolve reliable λ, η, ξ factors for a given set of conditions. This will provide base for the deformation-stress transformation and stress analysis of embankments.

9 ACKNOWLEDGEMENTS

The work was initiated as topic of dissertation for M.E.(Civil)by Shri B.J. Patel, Lecturer, Polytechnic and latter extended Ph.D. problem of Shri T.B. Desai. The author is grateful to Shri D.C. Bhrambhatt and Shri H.A. Lad, Drafteman of Deptt. of Applied Mechanics for valuable assistance.

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REVIEW OF EXPERIENCES OF GEOTECHNICAL
EXPLORATIONS FOR DAM FOUNDATIONS.

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1.0 INTRODUCTION :

Association of 30 years with exploration in geotechnical engineering has been basis to present a phylosophy of subsoil exploration for dam foundations. IS Codes on programme and methods of explorations, ASCE recommendations and many practices have been in vouge. Briefly a foundation designer needs 4 design details from geotechnical expert. They are :

(1) Zoning of area of similar subsoil problems in plan for a site.

(2) Profile or stratification (section) for each Zone.

(3) Properties & type of soil of each stratification for design & analysis.

(4) Ground water fluctuations.

These basic details are usually vauge, hidden or nonspecific in most of the subsoil exploration reports.

The objective therefore to obtain 4 design details must be emphasied in planning programme.

2.0 EVOLVE DESIGN FROM EXPLORATION :

By and large, the scenerio is that designer assumes probable problems/parameters from similarity of site with projects executed and proceeds with design of treatments/ structures/foundations. Visits by few experts to sites adds certain probabilities based on their remarks. The sand is loose and may liquify or soils are loessic and may collapse on saturation or soft clays appear sensitive etc. will be imposing constraints to adopt safety against assumed behaviour. As expert is sometimes authority, true but contrery data of investigations may have to be suppressed. If officer is prepared to face facts he may have difficulty to convince experts of repute. This background of anticipated troubles predetermines exploration programme. The constraints of agencies, equipments, personals, working conditions at remote sites, doubtful integrity of field engineers will decide outcome of exploration. The designers have hardly faith in data presented. The time taken by large exploration justified designers assuming parameters and proceeding with design and some times with construction which were to be confirmed safe by exploration i.e. design is rarely evolved on basis of scientific exploration in past. The report in annexture indicates that design assumptions are safer. The soil exploration has been force in many cases.

Now in present better facilities and less contrainsts environment, one must break chain to establish correct cycle, First exploration, then identification of critical parameters and problems for site based on exploration should lead to best economical and time saving design alternative. The soil exploration data shall be basis to identify weather shear, differential settlement, piping, liquifaction seepage etc.are likely. Individual openions or doubts could be evaluated on basis of investigations and if need be some special explorations be conducted to remove contradictions. For economy, safety and time saving sequence of preliminary exploration, analysis to establish critical aspects for site, reconfirming critical design parameters and selecting best alternative design/treatment must be restored.

3.0 DESIGN PARAMERS - SENSITIVITY ANALYSIS :

Soil exploration report will present digested data on four aspects of design details. The foundation design for shear, sliding, settlement, seepage, piping, liquifaction could be evaluated using sensitivity analysis of ± 10% errors in design parameters. For example taking + 10% of shear parameters stability of foundation slip circle changes from unsafe to safe value, it is desirable to recheck the exploration data on shear parameters. This will be special investigation on critical location in zone and layer by one or more direct indirect as well as laboratory and field tests to ascertain minimum safe parameter. Sensitivity analysis will restrict area of exploration for detailed study.

To site an example, based on profile, the shear parameters the safe bearing capacity is evolved. Using this bearing capacity if design indicates provision of raft foundations marginally sensitivly analysis could be carried out using 5% increase in cohesion makes footings feasible, rechecking the shear parameter for stress bulb zone only is advisible. Rechecking results of UCC by triaxial or plate load or pressurometer at critical point selected from preliminary data can not only provide economy but save time of construction as well. In short all sensitive parameter which can alter design/ground treatment need to be reevaluated by alternative or same method precisely.

4.0 RIGID & FLEXIBLE PROGRAMME :

It is quite common to adopt printed common specifications for exploration of site irrespective of the geological formation and geographical locations. Each consultant or organisation has a draft in which depth, number of bores, tests are entered for a site to suit budget, time and sometimes requirements. Such universal draft is essential for competitive bids and financial audit. This system includes many redudent tests and points from utility point. Also job is to be completed by contract till final report no analysis is feasible. Huge amount of sampling and testing will present a thick forest of data having many discripancies (SPT contradicting density -ϕ value or sand showing low K etc.). Once designers finds faults and looses faith he will go by worst, uneconomical empirical design.

This has to be changed by system in which each exploration programme is analysed and used to evolve further programme. Exploration will be flexible and will be evaluated for 4 objectives contineously. Here sesmic/Dyn cone penetrometer (50mm) could be used to cover 4 corners of the zone and centre. The depth versus N_c or seismic velocity is interpreted to evolve soil profile, soil properties Ground water, liquification potential by approximate empirical charts. This data is basis. To refine it one or more bores are used. The profile, properties, W.T. etc. are revised again. In this system testing sampling is specific and not mechanically SPT/sampling at 2m interval sampling is clays and SPT in sand will be used. The critical factors could be further checked by special investigations of specific layer by vane, pressureometer, permeameter, static cone etc. All the special exploration have specific objective to narrow down range of predicted design parameters or problems. You can eliminate unwanted SPT in clays or UD sampling in sands and K test in dry sand strata below building foundation. Exceptional costt sensitive sites, extra ordinary explorations by blasting, load testing, groutability tests as well as insitu density by neuclear probes are suggested to reconcile different behavioural predictions. Objective of each special exploration will be to predict parameters to predict performance as accurately as possible.

5.0 STATISTICAL AVERAGE PARAMETER :

It is common practice to compile results of number of samples of a stratum for all properties to derive statistical average parameters for design parameters. This could be misguiding.

For a stratification of a given zone, average coefficient of permeability (K) from results of tests on 20 samples gave range of 10m/sec to 10^{-3} cm/sec is absurd to evaluate seepage or groutability. Similarly shear parameters of soft to medium clay ranging from 1.0 kg/cm^2 to 0.4 kg/cm^2 for 30 undisturbed samples (based on UCC on samples of variable degree of saturation) as 0.56 kg/cm^2 would lead designer nowhere. A geo-engineer will have to discard results of distrubed samplea, dried samples, samples of low degree of saturation insitu (after reservoir is full it will be saturated) and samples having local failure plane etc. The acceptible data when average gives Cu = 0.48 kg/cm^2 against statistical average of 0.92 kg/cm^2. Such analysis based on sensitivity study of design procedure would justify the confirmation of Cu by vane or static cone test at location where subsoil is saturated now. Even results of specimens showing abnormal low or high density will have to be scrutinised by Engineering judgement. In short use of statistical averages widely used, even for large number of samples could be disastrous or uneconomical. It shall be replaced by geotechnical engineers judgement based on experience subject to confirmation by other test in laboratory or insitu.

6.0 CROSS CHECKING RESULTS :

It is cumbersome but essential to cross check all results of investigation of dam foundation. For clayey sand partly saturated, very high K or for sand with N_S = 40 blows dry unit weight of 1200 kg/m^3, clay with nat.water content equal to plastic limit cu very low illustrates need for cross checking and retesting. SPT less than 10 indicate low Relative density but density of sand insitu gave very high relative density. Former may pose problem of liquifaction for site at which latter indicates no liquification. In all marginal cases prototype field trial of blasting or grouting are justified as special check tests. We have come across soil log sand at dam site which did not accept cement grout and low N value of obra/Ukai sand did not liquify by sesmicity much more than expected under a insitu blasting tests.

A contineous exploration with systematic cross checking can permit repeat testing/alternative or extra investigation to provide designer a true design factor.

6.0 CONTINUING EDUCATION :

Learning from mistakes/wrong interpretations cannot be published from the possible victimisation by administration. CBIP/CBRI & national societies can hold a session at annual meetings to discuss experiences without naming project/person. This will provide continuing education and repeating silly mistakes again and again by different persons at different site could be eliminated.

CONCLUSIONS :

(1) The rigid preplanned blueprint of exploration for major projects needs drastic review. A flexible approach to evolve parameters by DCP/Geophysical methods to be confirmed by borings planned on basis of analysis of each bore will provide safer and economical designs.

(2) Each exploration programme must be analysed to sequentially evolve further programme/method of exploration in continueous process to evolve zoning of site, soil profile in each zone, properties of soil in each strata and G.WT. The preset tender programme provides vast jungle of data with wide ranges of values making a scientific selection impossible. Vast data collection takes so must of time & agencies that cross varifications or checking is also not possible.

(3) Soil exploration for major projects must guide design for economy and safety and not form annexture of a design as formality.

(4) Never use stastical averages of parameters in design. Use geotechnical skill to eliminate non representative/unreliable data to obtain reliable range. This range shall be reconfirmed in field by independent cross checking insitu test conducted precisely in limited number.

(5) All critical design parameters must be obtained by one test, checked by another and reconfirmed by field tests if it shows persistant analomics. Assuming lowest/worst need not be safe always in geotechnical engineering.

(6) A forum for discussing silk mistakes without names of project persons will help providing continuing education to dam Engineers.

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EVOLUTION OF EARTH DAM DESIGN - PERFORMANCE ILLUSTRATED BY CASE STUDIES

By:

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Ex. Dy. Director, CWPC

Visiting Prof. SVNIT, Surat.

Introduction:

Of Civil Engineering Structures, built by Engineers since independence, Earth Dams constitutes – largest, most complex structures with least scientific and maximum empirical design parameters.

Our Green revolution and temples of Nehru's dream are reality thanks to Earth Dams – around 5000 Nos.

No foreign technology

Least steel, cement and plant and machinery, maximum use of manpower. (1950-1990)

Basic design Inputs are Natural and based on Observations – Experience – Judgment

- **River Course, History**
- **Construction materials & its behaviour**
- **Foundation – Geology – Seismicity**
- **Natural forces – Earthquake, Floods, Drawdown**
- **Remote inapproachable sites (1958-1980) & meger history & data for sites selected.**
- **Case studies rarely published (Confidential)**
- **Investigation, Construction agencies not available – Department skill is only source.**

Challenges:

- **Assessment of variability of Input for Design.**
- **To predict parameters during life span of 100 to 300 years under unknown future environment.**
- **Factor of safety, factor of ignorance or inability to predict and lack of knowledge / experience based confidence.**
- **Controls of material, construction specifications, plants & compaction equipments handled by inexperienced changing technical staff of Department of irrigation.**

Continued…

- **Project planning, designing, execution, maintainance changes hands as period of execution from conception is 20 to 25 years.**
- **Project administration setup has not been able to pool the experience based data from thousands of dams built.**

WE HAVE FAILED TO LEARN FROM FAILURES & SUCCESSES.

Thanks To:

In spite of above nonscientific non-engineering design, planning, construction failure rate of 1 in 2000 or so is outstanding achievement of Earth dam Pioneers:

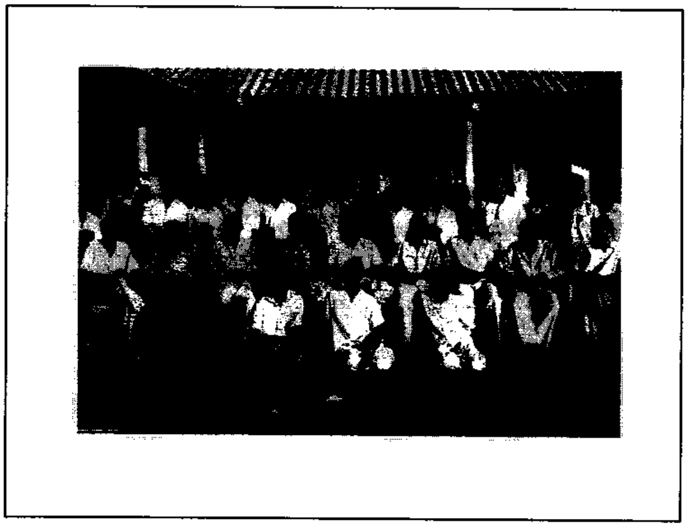
To remember few in field:

- **Dr. A.N. Khosala, Dr. K.L. Rao, Y.K. Murthy, Dr. C.D. Thatte (CWPC)**
- **G.G. Dhanak, M.N. Jathal, J.F. Mistry, M.U. Purohit (Govt. of Guj.)**
- **Academician: Dr. V.J. Patel, Prof. Govind Rao**
- **Politicians: Planning minister Shri Ashok Mehta & Pandit Nehru.**

1959-60
Brig. Bose
Prof Arsiwala
Earth Dam

Dr KL Rao
Dr Pars
Mr M Desai
JR Khanna

4



1961
Ukai,

(1969)
CS. MRS
group.
– 61 Hydroelectric
Projects
– Khurrani
–

5

Constraints:

- Assessment of forces, static, dynamic to predict properties of millions of cubic meter of soil used at different levels and parts over 10 to 20 years for the critical conditions over life of 100 to 200 years of dam
- Predicting behaviour of foundations for dams by extrapolation and limited history, geology, seismology and weathering under submergence
- Optimize cost inspite of oversafe design.
- Cater and adjust to State of Art of materials, observations during construction, technology, plants availability etc.

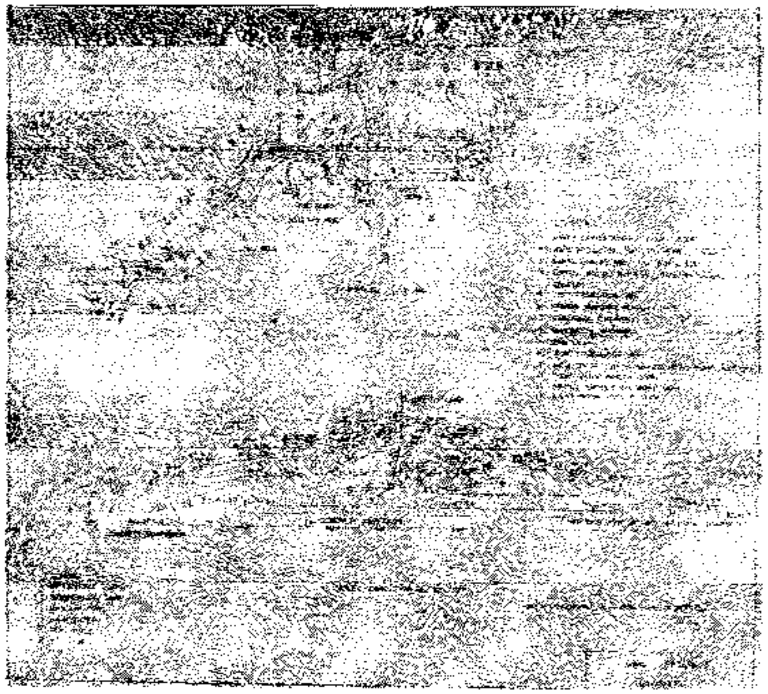
Concepts of "Design as you Construct or Proceed" was evolved by me. Also calculated risk in parameters, is unavoidable.

(Ref: Evolution of Design of river valley projects, journal of institution of engineers, Vol. 53 July 1973, Pg - 288)

e.g. Construct P.P. measured at Beas / Ukai was less than design ... reviewed, slope for reduction

Borrow area of Ukai dam reviewed to use 1 MCM of random fill soil, available from excavation of power channel. Section of dam revised.

Silt stone as core for Beas / Black cotton soil clods in Ukai.



Ukai Dam

History:

Though century old technology (Art), slow steady progressive development can be seen over past 60 years

Year	Global Max Height (m)	Indian dam planned	Remarks
1920	60	20	Oldest (100 yrs or so)
1970	280		Ekruk dam (Maha) Kishau (252 m)
1980		260*	Tehri (260 m)*

Inspite of technical, environmental, ecological, social political odds dams have started serving society in 2000. Beas and Ramganga dams have stood test of time (Seismic zones VI to X) Richter scale 8.6 earthquakes.

Evolution & Updating:

Pre 1960 dams planned with 4H : 1V slopes (Gangapur, Ukai, Panset etc.) have been replaced by 1.5 to 2H : 1V u/s slopes. D/s slopes have decreased from 3H: 1V to 2H : 1V. More than 5000 dams with hardly 3 major failures (Panset, Morbi and Tiddi) shows unusual success rates inspite of empirical approach, department execution, poor quality control and changing of In-charge Engineers over 20 -30 years period of execution.

Thin inclined core was introduced for dams in north India as cohesive soils are rare. The "Design as you Construct", philosophy permitted use of materials of compulsory excavations improved plants and technologies available during long period of construction.

Limitations:

The topo sheet, site selection, the horse back visit to evaluate site data, ~~by visit~~, visual observations and experience based judgment. Feasibility and economical viability for dams in 1960 – 70 were based on meger scientific data of survey, geology, construction materials, design and cost benefit ratio. The communication, contracting firms, expert services investigation facilities and foreign exchange for construction plants, as on today, cannot be dreamt. The credit of what serves us today goes to Public Works Department of Irrigation & CWC (now discredited as P.W.D.)

Ghaghar Dam (1963)

Sala~~d~~l Dam (1966)

Photos shows typical site investigation based on observations on typical pits, trenches, kotar & tunnels by Junior Engineer. The slopes of materials in field were angle of repose & cohesion was assumed low at saturation, drawdown conditions.

Design:

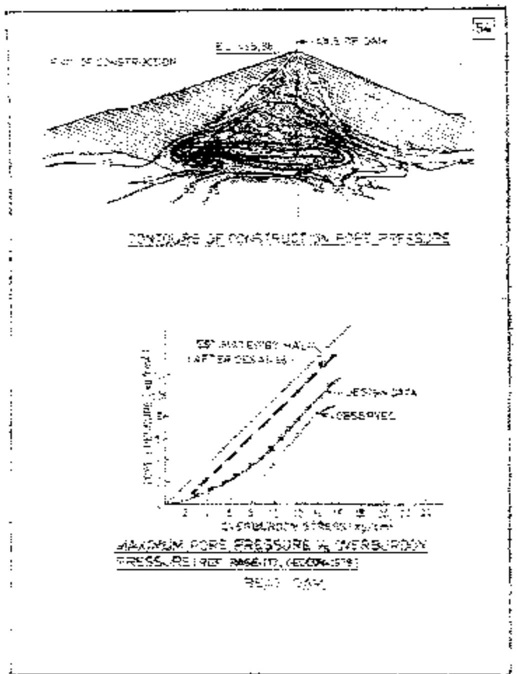
Major Design Criteria:

- **Stability based slopes of dam. Properties of materials of construction (30 - 40 M.Cubic in Ukai) critical in life span of 100 – 200 years. c - Ø - k – Compressibility. (Borrow areas)**
  - core - casing, Rock-fill, Filter – Construction, Steady seepage, drawdown.
- **Control of seepage through foundations & embankment – piping, sloughing, boiling, liquefaction**
- **Control of deformation w.r.t. construction, drawdown, over life period under static – dynamic states.**
- **Control of erosion by rains**
- **Sliding**
- **Over toppling by floods.**

Case Studies

Beas Dam:

There was shortage of impervious core material. Excavated silt stone in area after a rainfall crumbled to silty sand. It was adopted as core material after hot debates. Field observations provided a economical core material.

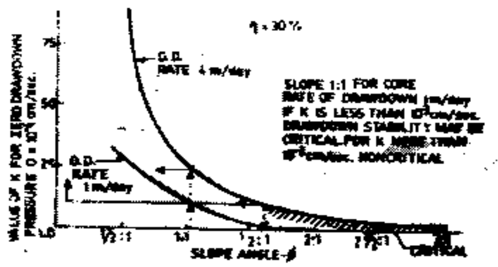


Construction pore pressure for design of dam is estimated by Hilf method. For Beas dam, construction of core consists of wide range of soils, different mode and degree of compaction (Desai '67). Figure shows pressures, predicted and adopted for design '68, '93. (Desai) Observed P.P. are also shown. The design was very conservative and during later half redesigned based on observations.

Ukai, Damanganga dams in first phase confirmed observed construction P.P. are 60 to 70 % of theoretical.

<u>Beas Dam</u>

Rapid Drawdown:



STABILITY OF UPSTEAM SLOPE DRAWDOWN CONDITION

For cohesive core & semi-pervious casing dams, rapid drawdown condition is critical. The rapid state is undefined and hence criteria applied to all cases. Author has studied porosity rate of drawdown m/day, slope of core of dam upstream with critical coefficient of permeability k value. If actual k is higher than critical value drawdown stability is not critical.

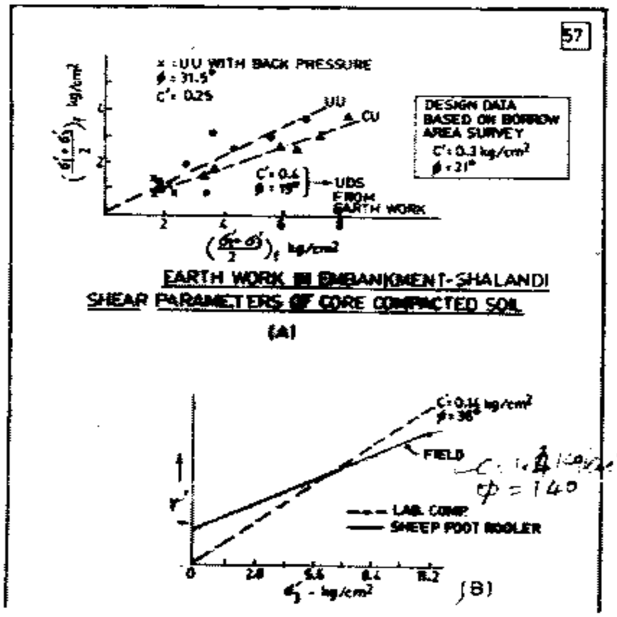
e.g. For earth dam η = 30 %, core slope 1:1, rate of drawdown 4 m/day, critical k = $25x10^{-4}$ cm/sec. If actual k for soil is higher, stability of drawdown condition is not critical.

Shear Parameters – Field Compaction:

The field compaction by different plants gives different structures for soil in field compared to samples compacted in laboratory.

Our R & D shows:

The field parameters gives different parameters for different soils & techniques of compaction. Thus factor of safety estimated could vary actually.



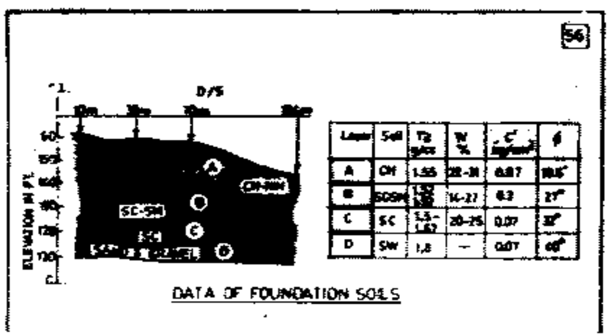
SM Soil at Beas – Effect of Compaction by Sheep foot roller

Shear Parameters:
Lab compaction vs Field Compaction
Samples compacted at OMC to MDD was saturated to evolve c' - Ø' for stability analysis.

Actual constructed fill samples CH of Shalandi and SM-ML of Beas were evolved by author (Desai'75), shear parameters inspite of wide range of soils compacted to range of densities, probably due to different structure showed higher parameters.

(1968)

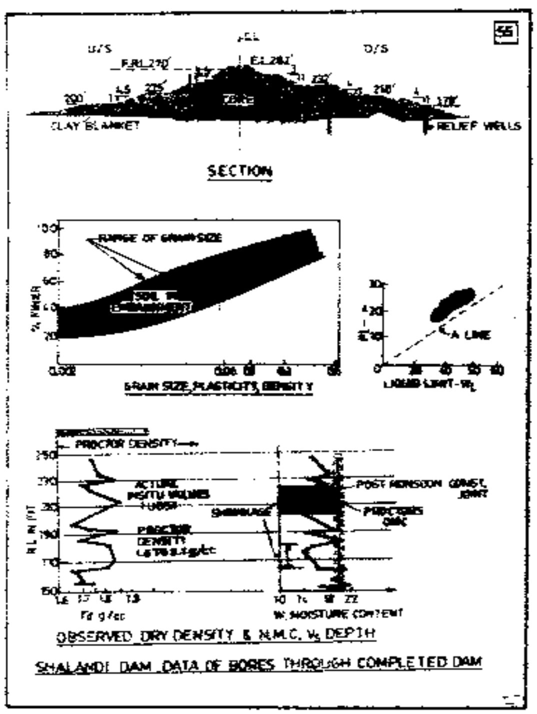
Need for Personal Observations in Foundation studies:



Field engineers observed different subsoil profile in foundation than adopted in design. Executing agency reported low c - Ø, high k and high SPT N values: Recommended Flattering of slopes and grouting. CSMRS checked data by 3 bores: Profile of foundation is shown. Stability satisfactory flattening rejected. No Grouting

Shalandi dam Leakage in core:

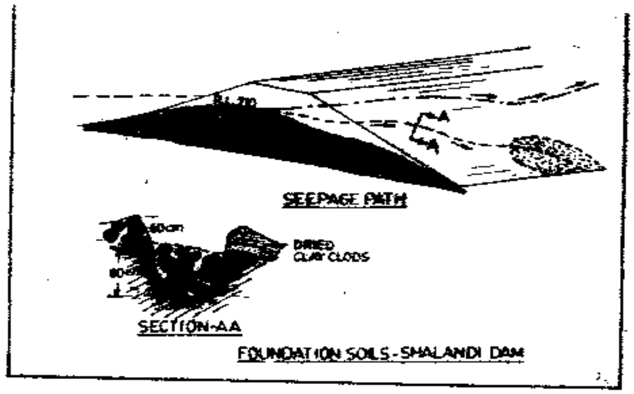
Studies for foundation adequacy by field observations brought out cause of seepage in dam.



Field Control:
Poor quality control for Departmental work is taken care by factor of safety in Design.

Typical Dam: Shalandi, Soil grain size, plasticity range, proctor density 1.6 to 2.1 g/cc, OMC 19 ± 1 % are shown in Fig.

Field check of constructed earth fill shows ρd 1.65 to 1.85 g/cc, moisture varied from 10 to 22 % (Depth 100 Ft.) Drinking post monsoon located – 200 to 220' in a Bore. It explained seepage and dampness on D/S. Path variable as rain cuts geometry is zigzag. Treated by self curing + heaving. Filling of dam in steps 2 – 3' / year & watching downstream.

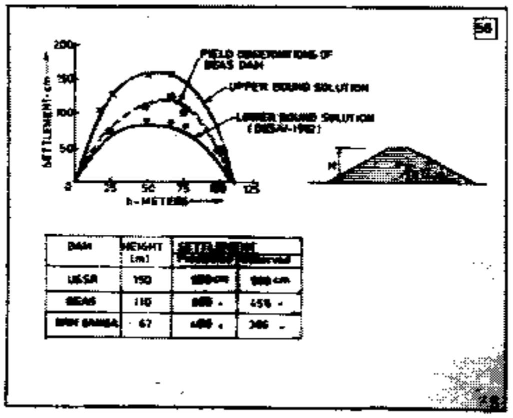


At RL 210' drilling (dry) showed heavy loss (k > 10^{-1} cm/sec). Rain cut filled by clods and covered by earthwork d/s Relief wells. Internet piping in rain cuts: Low head by filling 1 m/year, 40 % heave of CH clods sealed rain-cuts. D/s patches dried.

Vertical Displacement in body of Dams:

Displacement: vertical, u/s – d/s, axial.

A quick assessment of magnitude was made by research based on data published for some dams (well instrumented & observed.)



Theoretical methods of predicting displacements models based on observed good case studies Author (1982) is adopted. The tend is typical, Maximum vertical displacement is observed at Mid-height of Beas dam. The observation of Beas dam confirms trend and values. Such empirical model can provide Vertical, Horizontal and Axial movements to plain instrumentation and finite elements grids.

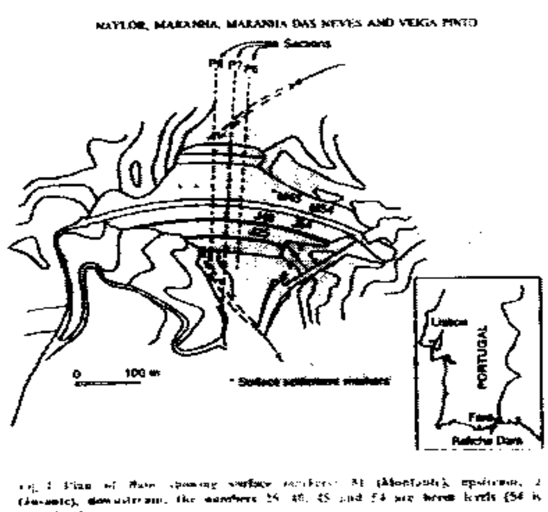
Beas Dam, H = 110 m, 16m (max)

A) Vertical St = ~~160 cm (max)~~, 110 cm (min), observed 120 cm, 1.2 m

B) Axial η = 0.025 h, max 1.25 m,

C) (U/S – D/S) Horizontal Core 1.5 % H to Casing 3 % H @ mid height, i.e.
St = 1.5 m core & 3.0 m casing.

Recent data of Beliche Dam (1997):



Plan

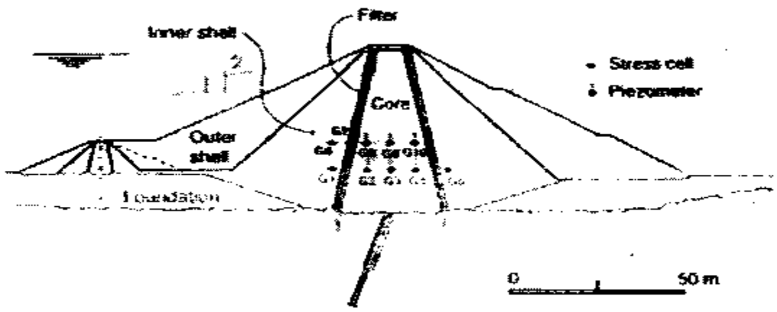


Fig. 2. Section P7 showing stress cells and piezometers

Typical Section

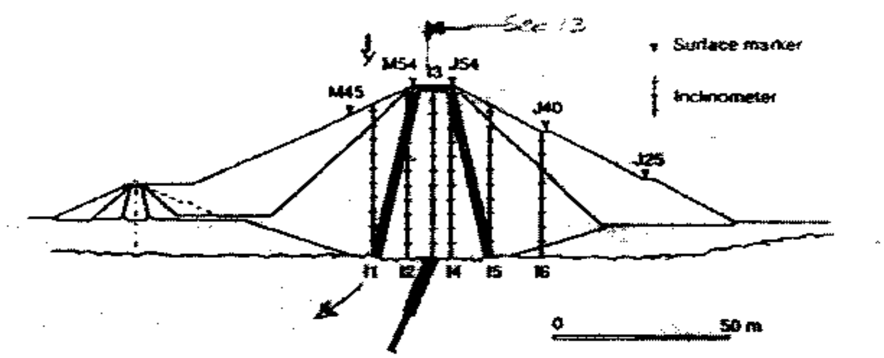
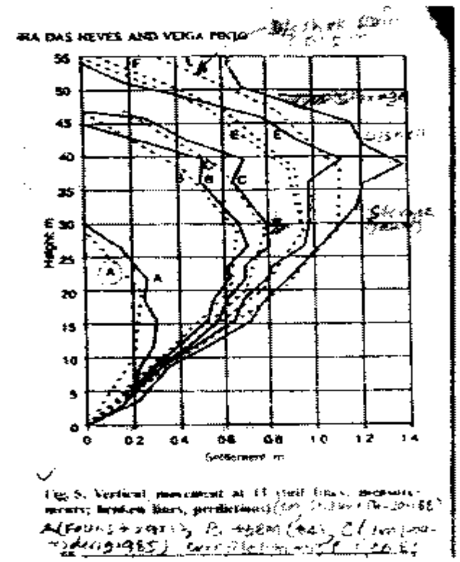
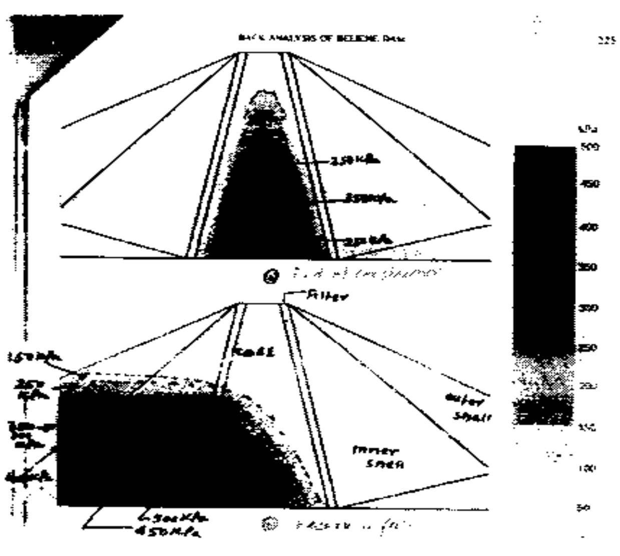


Fig. 3. Section P7 showing inclinometers and surface settlement markers

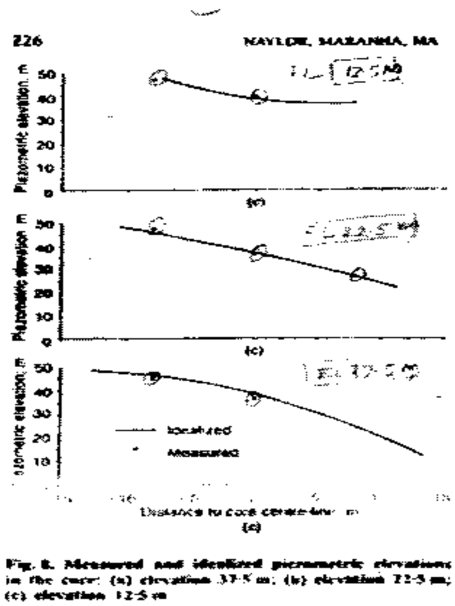
Typical Section Instrumented



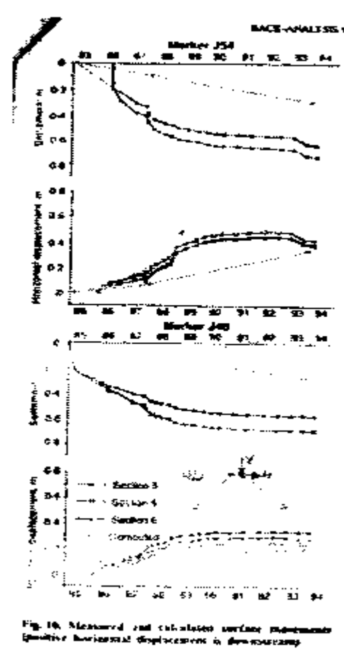
Vertical movement at Sec. 13,
(Broken line - Predicted, Dark line – Actual at reservoir filling)
Note: Maximum displacement is at mid height, effect of storage on Stv



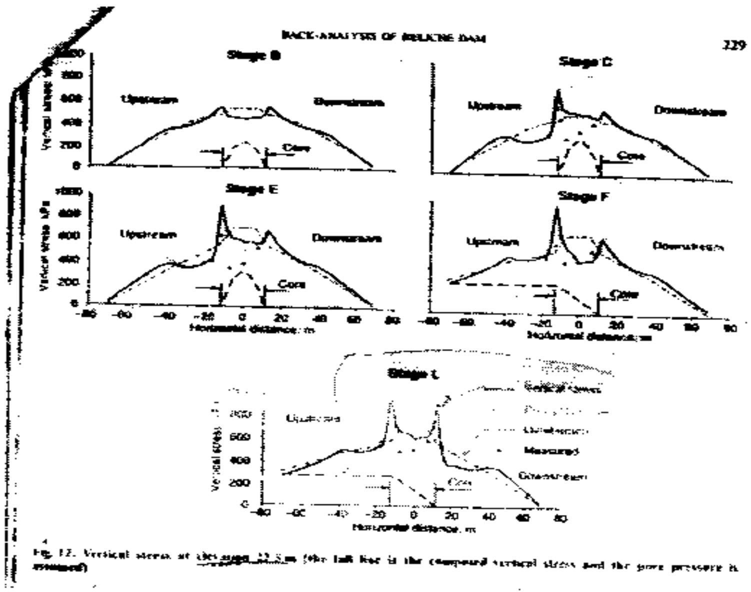
Pore water pressure in Core of Dam
(a) End of Construction, (b) Reservoir full
Note: P.P. values / filter action



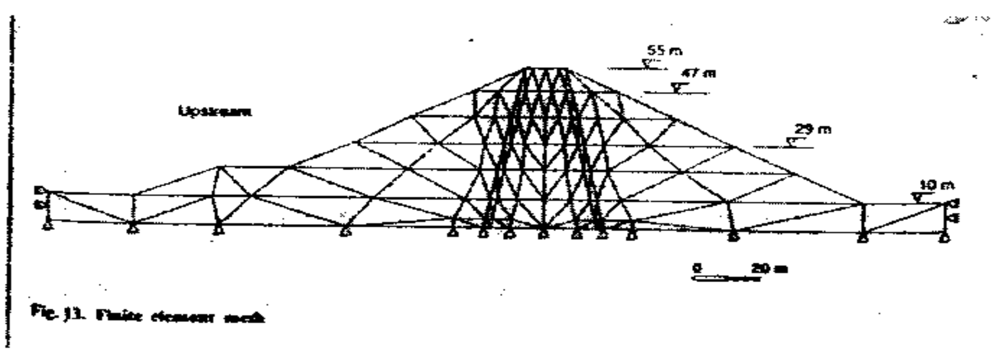
Pizometric deviation for El 12.5, 22.5, 37.5 m



Surface Movement (Horizontal to D/S)



Stress Vertical at El 22.5 m



Design Finite Element Mesh.

Table 1. Beliche Dam critical state model parameters

	E: MPa	ν	c_{cs}: kPa	ϕ_{cs}: degrees	χ	κ	σ_{s0}: kPa
Core	30	0·2	30	25	0·006	2	40
Filter	45	0·2	20	40	0·006	2	45
Inner shell (dry)	45	0·3	22	41	0·006	2	40
Inner shell (wet)	21	0·3	10	34	0·01	2	25
Outer shell (dry)	90	0·2	35	50	0·0025	2	80
Outer shell (wet)	80	0·2	25	48	0·0025	2	60
Foundation	60	0·28	23	35	0·005	2	50

Soil Parameters

BACK-ANALYSIS OF BELICHE DAM 231

Table 2. Measured and calculated total stress invariants at end of construction (Stage E).* (E ≈ 55 M)

Load cell	Zone	Measured		Calculated ($\sigma_1+\sigma_3$)	$\sigma_1-\sigma_3$
		σ_p: kPa	σ_q: kPa	σ_p: kPa	σ_q: kPa
G1	Inner shell	637	881	497 (561) [629]	249 (302) [495]
G2	Core	539	224	639 (456) [442]	301 (392) [276]
G3	Core	537	313	631 (481) [468]	199 (432) [334]
G4	Core	443	300	619 (459) [442]	260 (396) [278]
G5	Inner shell	1070	1000	585 (539) [620]	370 (397) [499]
G6	Inner shell	304	186	338 (402) [431]	138 (417) [502]
G7	Filter	469	295	630 (517) [477]	483 (478) [578]
G8	Core	327	35	499 (322) [330]	253 (256) [243]
G9	Core	313	193	481 (366) [352]	187 (343) [267]
G10	Core	—	-	468 (322) [329]	218 (257) [245]

*( ) LNEC and [ ] Swansea predictions (Naylor et al., 1986).

1 m ponding collapse, settlement