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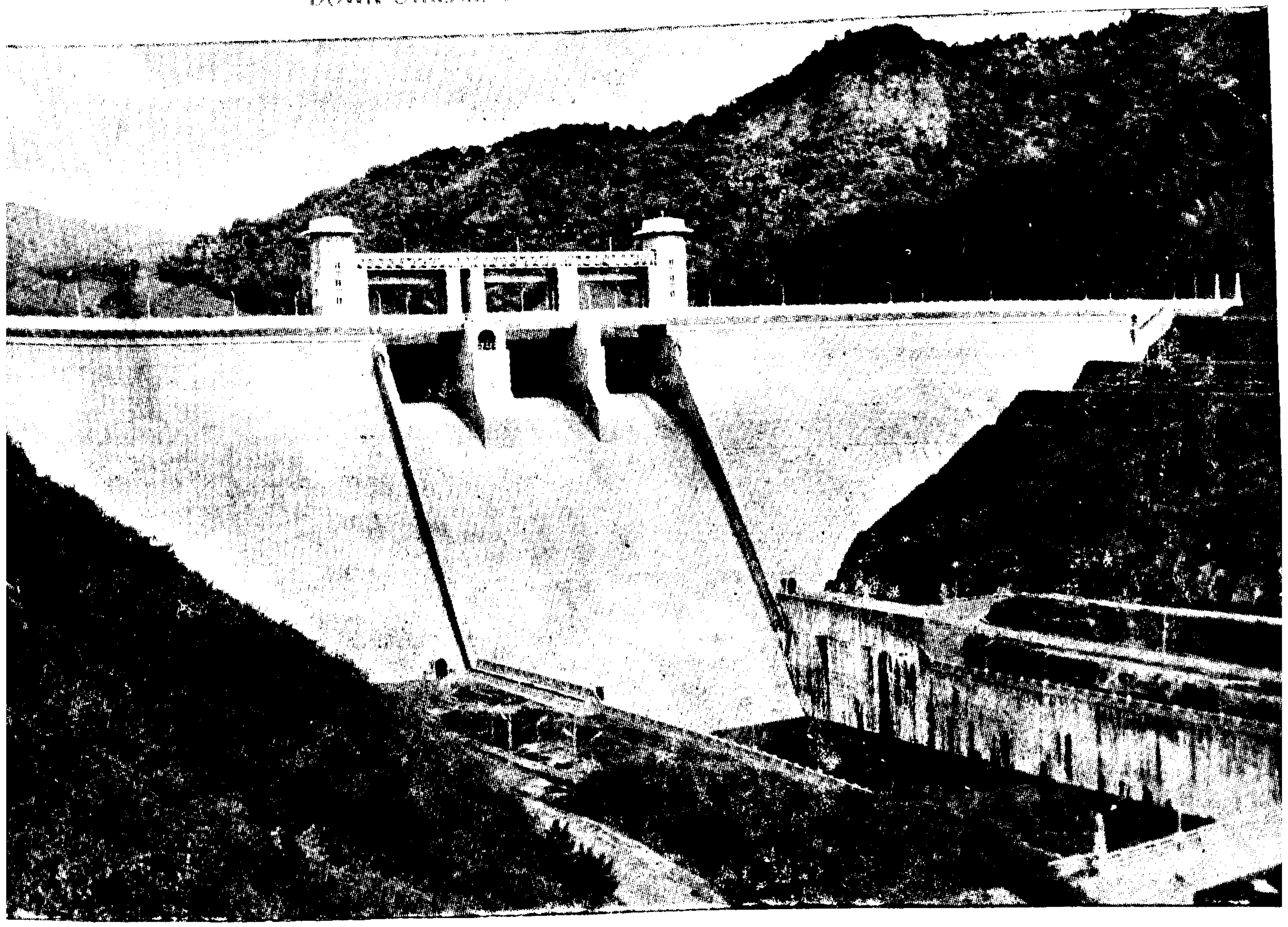
DOWN STREAM VIEW OF PARAMBIKULAM DAM



SILTING OF RESERVOIRS

By

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INTRODUCTION

Reservoir silting results from the deposition of stream borne soil or sediment when the transporting power of the stream is suddenly diminished by flowing into the quiet or impounded waters of a reservoir. It has been said that the history of a lake or a reservoir is the history of its death. Reservoir silting is to day a serious problem not just because sediment is deposited, but because it is deposited in such quantities as to cause serious economic and social losses. The primary cause of this condition is *accelerated soil erosion.* The rapid depletion of reservoir storage capacity resulting from soil erosion causes unusual damage to the tune of many crores of rupees, when measured simply by depreciation in investment.

EFFECTS OF RESERVOIR SILTING

Silting decreases the storage capacity of a reservoir which prevents it from rendering the full services for which it was designed, In reservoirs used for regulatory storage to maintain a high rate of primary power production, a decrease in capacity is directly reflected in reduction of primary power. In addition, complete silting of the reservoir, will allow the course particles to pass through turbines which may cause abrasion thereby damaging the turbine. Sedimentation damage to irrigation reservoirs is measured in terms of the value of water wasted over the spillway that could have been retained and used for crop production. Deposition of sediment in the deeper parts of the reservoir results in a greater surface area exposed for a given storage volume, which may cause a critical loss of water through Increased evapotranspiration. In flood control reservoirs, reduction of capacity will be reflected in the size of flood which the reservoir can fully control and therefore, sedimentation will increase the frequency and magnitude of flood damage down-stream.

Damage to water supply reservoirs is represented by reduction of the storage capacity below the minimum required to safeguard the continuity of a supply fully adequate for the present or estimated future needs. Any reduction by silting represents a direct damage which involves both an accumulating loss of service value and future replacement cost. In recreation reservoirs sediment creates conditions unfavourable to fish life and causes swamping in the upper end of the lake and on the shores, thereby decreasing esthetic values and often property values. Navigation reservoirs are damaged both by shoaling of ship channel, which necessitates frequent costly dredging operations, and by the loss of storage capacity required for regulating low water flows.

RATE OF SILTING

The rate of silting of a storage reservoir may be experessed by the relation.

$$C_1 = \frac{E\ Q_s}{C} \qquad (1)$$

Where C_1 = annual silting rate or capacity loss, in percent per year.

E = trap efficiency, or incoming sediment trapped in per cent.

Q_s = annual net sediment production from the drainage area in acre-feet per year.

C = original reservoir storage capacity.

The trap efficiency of a reservoir depends upon a number of factors. Among these are the ratios between storage capacity and inflow, age of the reservoir, shape of the reservoir basin, the type of outlets and methods of operation, the size characteristics of the sediment and the behaviour of finer sediment fractions under various conditions.

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One of the earlier studies of trap efficiency was made by Brune and Allen (1) who developed a curve (Vide Fig. 1) relating percentage of eroded soil caught in the reservoir with capacity per square mile of drainage area (C/W ratio). The trap efficiency values given in this curve, however, are necessarily low because they are based upon rates of erosion rather than rates of sediment production to the reservoir. Subsequently Brown (4) developed a curve relating C/W ratio and true trap efficiency. This curve shown in Fig. 2 is represented by.

$$E = 100\left(1 - \frac{1}{1 + 0.1\frac{C}{W}}\right) \qquad (2)$$

Where C/W = reservoir capacity in acre-feet per squrae mile of drainage area, and E = true trap efficiency.

From this curve it can be seen that the field observation points scatter very much. The main reason for the spread in the reservoirs having the same C/W ratios may have different capacity inflow ratios. It has long been recognised that capacity inflow (C/I) ratio would be a more accurate index of trap efficiency than the capacity watershed ratio (C/W) which has heretofore been so widely used. The main objection in the past to the use of C/I ratio has been that records of inflow are lacking for some existing reservoirs. A plot of trap efficiency in percent against capacity inflow ratio prepared by Brune from study of 41 reservoirs is shown in Fig. 3.

Reservoirs with a C/I ratio of 1 or less may be classed as seasonal storage reservoirs, and those with a ratio greater than 1 as hold over storage reservoirs. The C/I ratio is a combination of the C/W ratio and annual inflow. The C/I ratio in acre-feet of capacity per acre-feet of annul inflow may be obtained by dividing the C/W ratio, in acre-feet per square mile of drainage area, by the annual inflow, also in acre-feet per square mile of drainage area. Reservoirs may have very different C/W ratios and annual inflows, yet have the same C/I ratio, and the same trap efficiency, other factors being equal.

For individual reservoirs, curves can be drawn correlating trap efficiency with detention time. A curve prepared for imperial Dam Reservoir, Arizona is shown in Fig. 4. Such curves are quite satisfactory for specific reservoirs, since other factors such as sediment characteristics, shape of the reservoir, and method of operation tend to remain constant. Churchill (2) taking into account both detention time and velocity of flow through a Reservoir, has developed a "Sedimentation index" which represents the period of retention divided by mean velocity. His curve, relating trap efficiency to the sedimentation index for several TVA reservoirs, is shown in Fig. 5. Although this curve is very satisfactory, information such as period of retention and mean velocity are not readily available for most reservoirs.

Annual sediment discharge (Q_s) is generally estimated either from suspended load records or from reservoir sedimentation surveys (7, 8). In converting suspended load records, in tons, to volume of sediment, it is common practice to assume a value for the average specific weight of deposits at the end of the period for which the capacity loss is to be estimated, such as 50 years or the calculated useful life (i.e. 60 percent of the original capacity).

DISTRIBUTION OF RESERVOIR DEPOSIT

Because of the continuous increase in flow section and consequent decrease in velocity which occurs when a stream enters a resevoir, the corresponding reduction in transport capacity of the flow may be expected to result in a progressive deposition of its sediment load from the coarsest at the head to the finest at the dam. In addition, however, a typical primary pattern of deposits will evolve with time, with secondary variations depending upon the conditions of reservoir operation and on the nature of sediment load.

At the head of the reservoir, the coarser portion of the load will be deposited. As depension continues, the resulting delta front will move progressively down into the lake, whereas the top-set beds will form progressively upstream as shown in Fig. 6. The greater the percentage of coarse sediment, the larger will be the volume and extent of delta, and hence the larger the proportion of material deposited in the upperlevels of the reservoir. However, the heavy seasonal drawdown which is common in most irrigation and many multipurpose reservoirs causes the formation of multiple delta fronts. Thus the greatest thickness occurs not in the highest levels of the reservoir, but near the point of lowest annual drawdown.

The distribution of sediment below the delta area depends to a great extent upon the characteristics of the finer portion of the sediment load. As a general rule, progressively smaller sizes of material will be deposited beyond the delta front resulting in a gradual downward slope of the reservoir bed. If the stream carries an appreciable

wash load, much of this material may not settle even though the cross sectional area of the stream increases. Furthermore, the suspension may not mix completely with the clear water of the reservoir because of its difference in density. Instead a gravity underflow commonly known as " density current" may result which will move through the entire reservoir.

Fig. 7 shows diagramatically the formation of a density current at the upstream end of a reservoir. The conditions at the so-called plunge point, where the river inflow disappears, beneath the surface of the reservoir, are especially difficult to define. Intense local mixing at this point is due to the losses involved in the change of momentum as the velocities of rivers are reduced to much lower velocities of the under flow. Usually the amount of energy available is not sufficient to produce complete mixing; however, the density current becomes diluted and the volume of underflow per unit time may be larger than river inflow, which causes a reservoir circulation as shown in Fig. 7.

The quantity of sediment transported by density currents is of great practical importance. Partial control of sediment deposition can be achieved by designing and operating gates and outlets in the dam to withdraw water of higher than average sediment content. Density currents probably rarely attain an average velocity of more than 1 mile per hour in passing through a reservoir. The most reliable information now available (5) indicates that density currents may develop and persist with very low concentrations of fine sediment. The thickness of muddy density currents may vary from a few inches upto, perhaps, 100 feet or more. From the results of density current observations in Lake Mead, Bell (3) estimated that about 232,000,000 tons of solid matter was brought into the lower part of the lake by these currents between the date of closing of the tunnel outlets, May 1, 1936 to May 20, 1941. This is close to one-third of the total sediment load brought into the lake during this period. Had suitable outlets been installed in Boulder Dam and had it been possible to operate them with a view to releasing this sediment, probably 75 to 90 percent of it could have been passed through the dam and this would have extended the useful life of the reservoir by 35 to 40 years.

From the laboratory study done by R. T. Knapp and H. S, Bell at the California Institute of Technology, it was postulated that

1. Density currents can be withdrawn through openings in the dam.

2. For release of density currents, large openings are probably not so desirable as against a great number of small openings

3. The data now available are not sufficient for fomulation of any general rules on the number, spacing, and distribution of outlets and control works required for most effective venting of density currents, and

4. An alternate method of venting underflows, although only tried on a model scale, consists of placing a curtain or a screen in front of the dam. Such a metal of thin concrete curtain in front of the dam would force wastage of sediment laden water at the bottom of the reservoir rather than of the water at the surface (Fig. 8).

SILTING CONTROL

All the methods proposed for the control of reservoir silting may be classified into six groups :

1. Selection of reservoir site
2. Design of reservoirs
3. Control of sediment inflow
4. Control of sediment deposition.
5. Removal of sediment deposits

and 6. Watershed erosion control.

The first two methods disregard the possibility of changing the rate of sediment output from the drainage area, or the control of deposition in the reservoir, but represents an attempt by controlling the location and size of the reservoir to assure the longest possible useful life. The feasibility of reducing rates of sediment production from the alternate drainage basins by erosion control measures, including land purchase and reforestation should be investigated. Choice of alternate sites is primarily confined to smaller reservoirs used for water supply, recreation, etc. An adequate C/W ratio based on reliable estimates of sediment inflow and characteristics of deposition should be provided to afford an economically sound period of usefulness.

The third, fourth and fifth methods accept a fixed rate of sediment production from the drainage area, as well as fixed size for the reservoir; but attempts, by various means, to prevent all or part of the sediment from being permanently deposited in the reservoir. Three principal methods have been used to control the inflow of sediment to a reservoir.

1. Settling basins (debris basins, sedimentation basins) on flowing streams reduce the velocity and cause deposition of atleast the coarser sediment. Two important points to be considered in the case of a debris basin are that maintenance must be provided inorder to keep outlet works free from clogging and that overflow waters must be returned to the original channel bed over non-erodible slopes or structures; otherwise the amount of materials eroded by desilted waters may almost equal the amount of material caught by the debris basin. Settling basins usually cost more per unit of storage than the reservoirs they protect.

2. Vegetative screens consist of dense plant growth through which sediment laden water must diffuse and flow with reduced velocity that causes deposition. Vegetative screen in certain cases, may be objectionable because of the large quantities of water consumed by evapo-transpiration.

3. Off channel reservoir, any by-pass canals and conduits are advantageous where topographic conditions are favourable and use of the entire stream flow is not required. If diversion into the reservoir can be controlled, the heavily sediment-laden flood flows can be by-passed.

The sixth method attempts through control measures in the watershed, to reduce the quantity of sediment that will reach the reservoir. The most effective means of preventing surface erosion is to protect the land with a permanent vegetal cover to the greatest possible extent. Forest shrub and grass cover decrease direct surface run-off and increase infiltration. Furthermore, it inhibits the formation of rills and gullies which not only contribute large quantities of sediment but also serve as natural flumes to carry the entrained particles of soil off the land slopes. In places where permanent vegetal cover cannot be provided, methods like channel barriers, debris basins, water spreading works, gully control structures, contour cultivation and stream bank stabilization will substantially reduce sediment production.

CONCLUSIONS

Six methods are enumerated for the control of silting of reservoirs. These six methods represent three philosophies of approach to the problem of control, namely by controlling the location and size of the reservoir to assure the longest possible useful life, by attempting to prevent all or part of sediment entering into the reservoir and through control measures in the watershed to reduce the quantity of sediment that will reach the reservoir. A sound policy of reservoir silting control requires impartial consideration of all those three philosophies, and selection from various methods available of those that may physically be possible and economically feasible under any particular set of conditions. A careful weighing of the advantages and disadvantages of the various methods before the reservoir is definitely located and designed is most likely to lead to the best solution of the problem.

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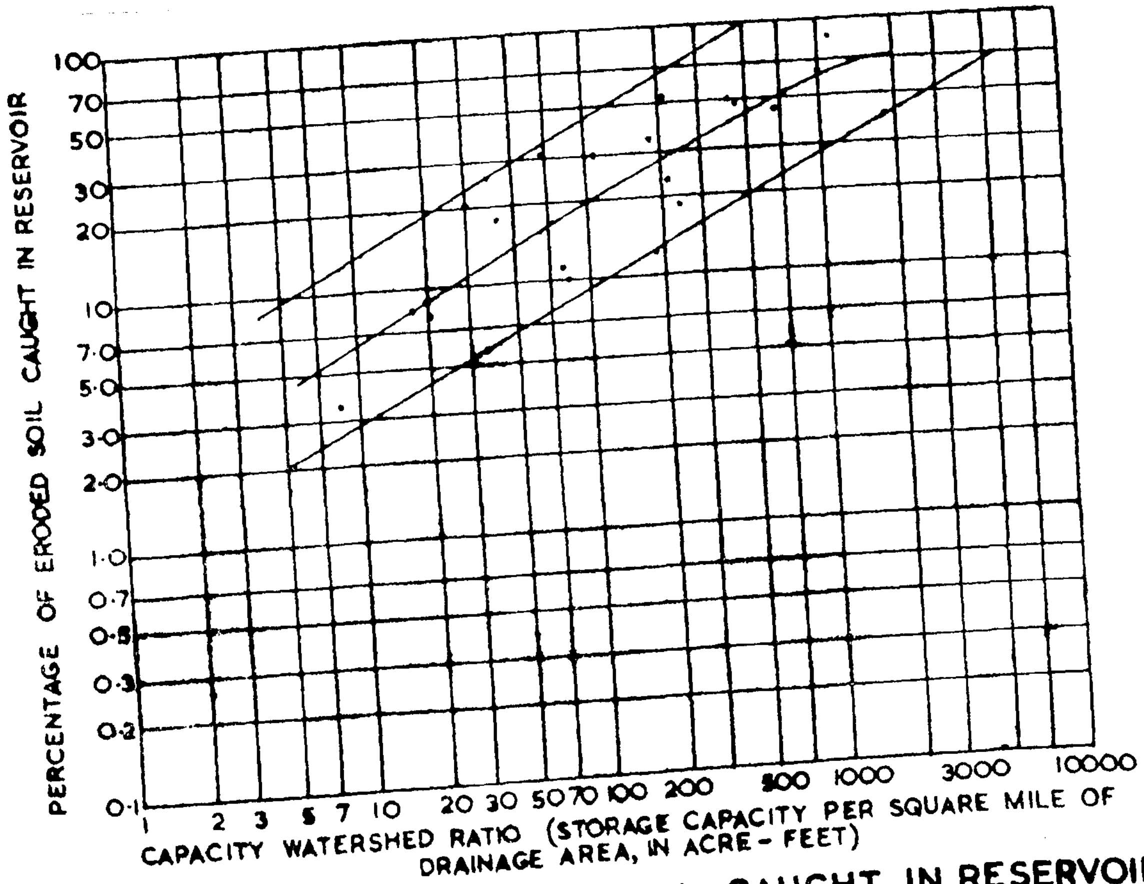

FIG.1 PERCENTAGE OF ERODED SOIL CAUGHT IN RESERVOIR AS RELATED TO CAPACITY PER SQUARE MILE OF DRAINAGE AREA.

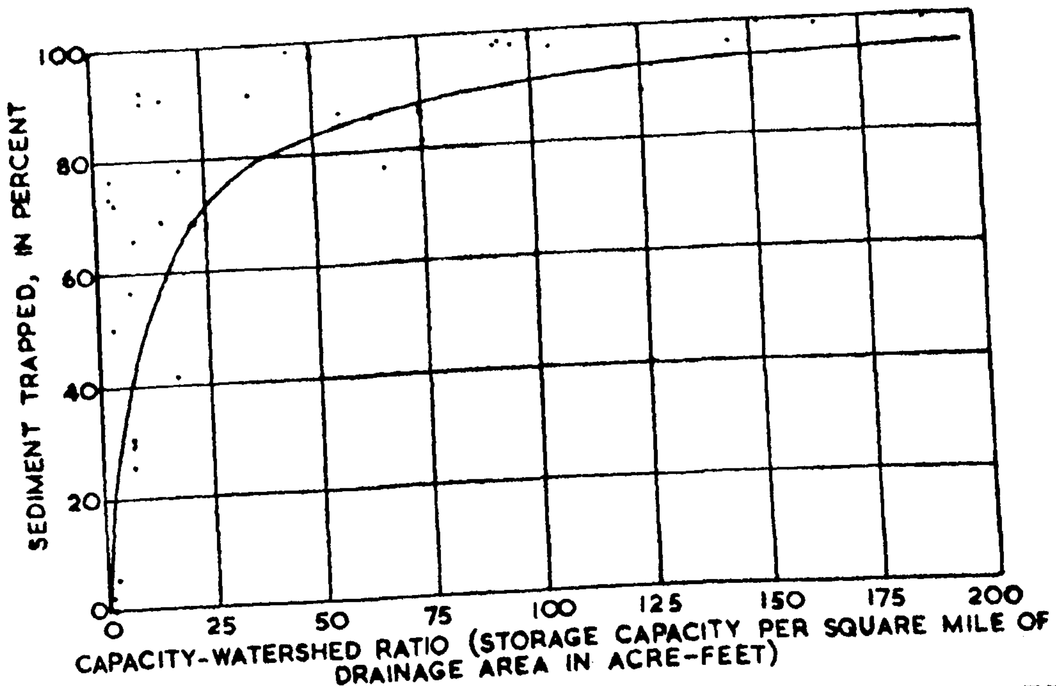

FIG.2 TRAP EFFICIENCY AS RELATED TO CAPACITY-WATERSHED RATIO.

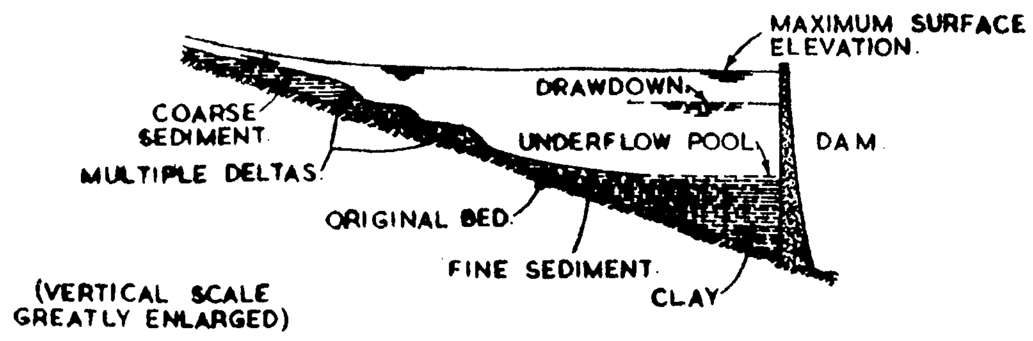

FIG. 6 SCHEMATIC REPRESENTATION OF SEDIMENT DEPOSITION IN RESERVOIRS.

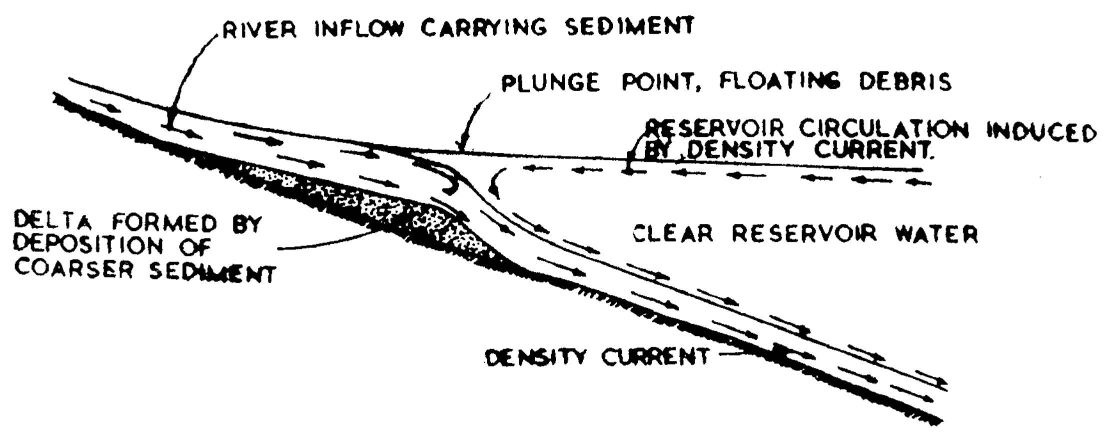

FIG. 7 UPSTREAM END OF RESERVOIR SHOWING FORMATION OF DENSITY CURRENT

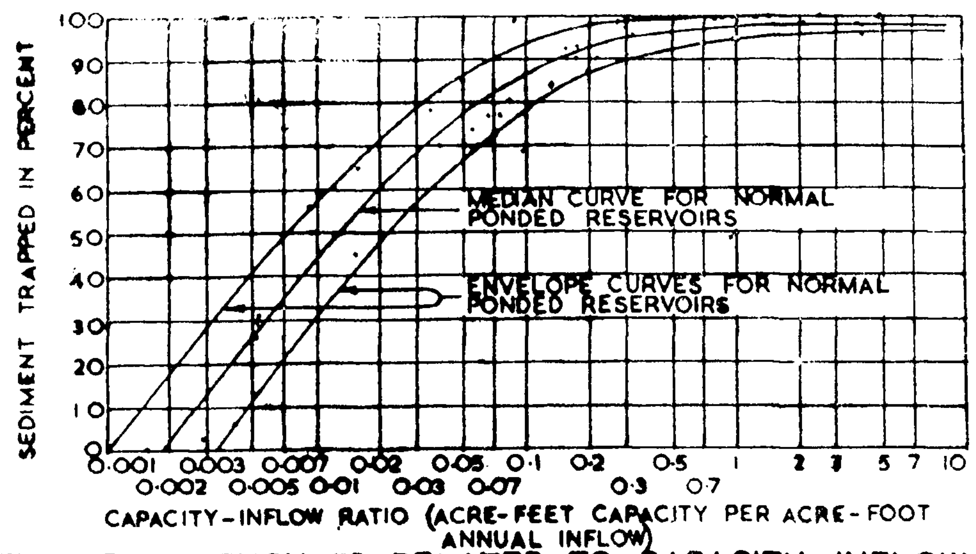

FIG 3 TRAP EFFICIENCY AS RELATED TO CAPACITY-INFLOW RATIO, TYPE OF RESERVOIR, AND METHOD OF OPERATION

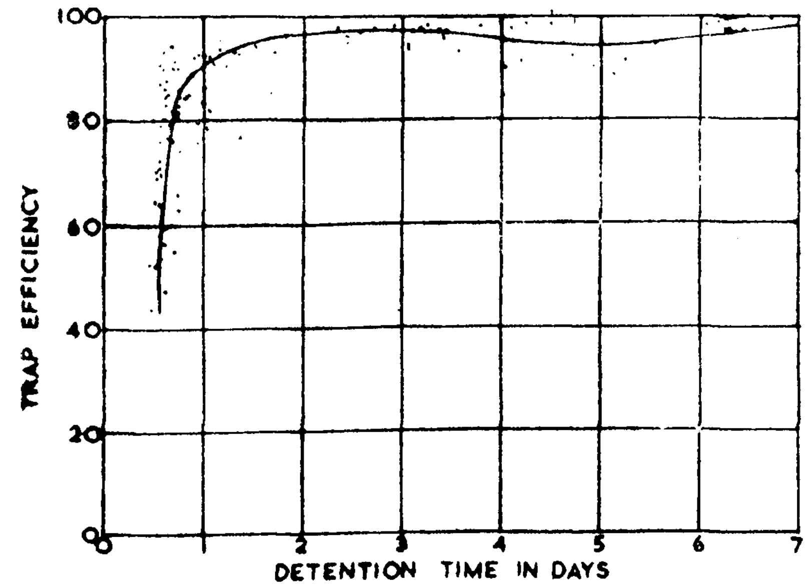

FIG. 4 TRAP EFFICIENCY AS RELATED TO DETENTION TIME, IMPERIAL DAM RESERVOIR, YUMA, ARIZONA

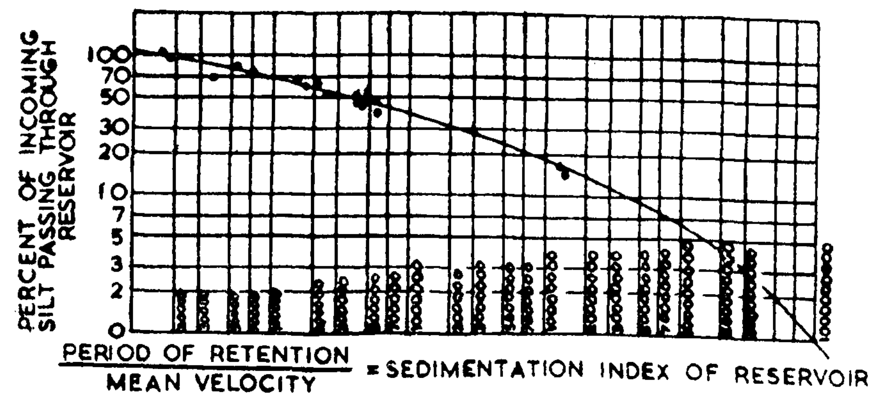

FIG. 5 PER CENT OF INCOMING SILT PASSING THROUGH RESERVOIR AS RELATED TO SEDIMENTATION INDEX, T V A RESERVOIRS.

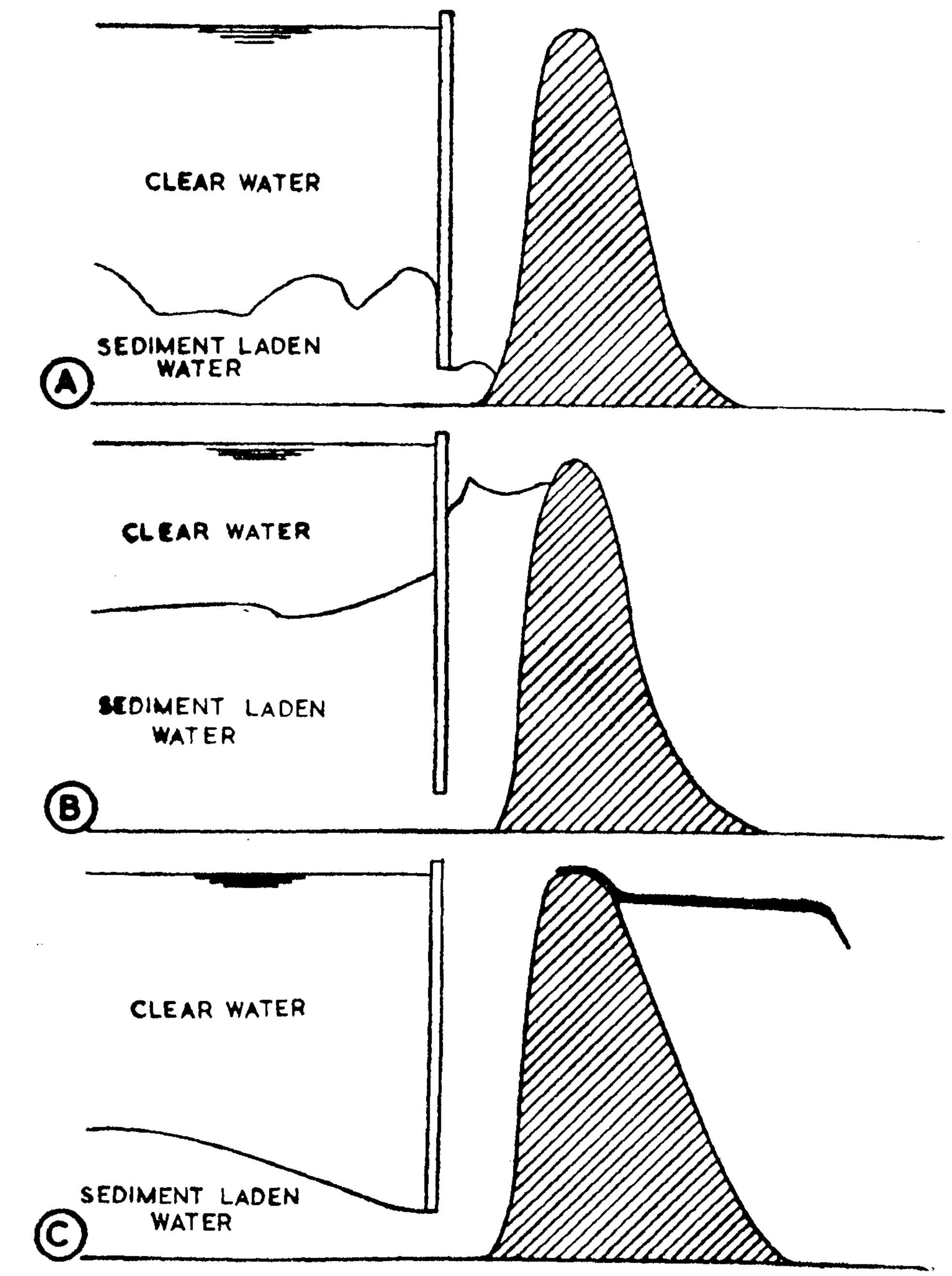

FIG. 8 A CURTAIN-TYPE OUTLET IN ACTION: A) AS TURBID DENSITY CURRENT REACHES CURTAIN, CLEAR WATER IS BEING FORCED OVER SPILLWAY; B) MUDDY FLOW CLIMBS TOWARD SPILLWAY BEHIND CURTAIN; C) MUDDY WATER DISCHARGING OVER SPILLWAY, WHILE CLEAR WATER IS HELD BACK BY CURTAIN.

A PLEA FOR THE USE OF 'ULTIMATE STRENGTH DESIGN' PROCEDURES.

By

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SYNOPSIS.

Critical examination of the allowable stress concepts is presented and load factors are advocated. The need for adoption of quality control on concrete is emphasised.

It is shown that the yield stress for steel and the cylinder strength for concrete are inadequate as data for the design of structures. Data on the behaviour of structural components and structures themselves are to be correlated with stresses.

Elasto-plastic methods of analysis are advocated and the need for research in this direction is indicated. There is scope for improvement in the I. S; 456-64.

Future trends are predicted and certain conclusions on the philosophical approach to the problem of concrete construction are drawn.

1. INTRODUCTION.

The field of structural engineering can be divided into the following areas of specialisation : (1) Planning (2) Analysis (3) Design and (4) Construction.

Any procedure which is rational, realistic and economical in any or all of these areas will be accepted by the engineering community and the rest will be rejected or superceded in course of time. This search for acceptable procedures has been the constant endeavour of structural engineers all over the world for the past sixty years and a better understanding, of the behaviour of reinforced and prestressed concrete structures under working and collapse loads has been obtained. Important conclusions that have been derived from these research activities will be presented and the future trends will be predicted.

2. LOAD FACTORS VS STRESS FACTORS.

Stress is not a directly measurable quantity while loads and strains can be measured. In structural engineering, we deal with large loads and micro-strains. Unless sophisticated instruments are available, micro-strains cannot be measured accurately. This logically leaves at the disposal of engineers loads only as a measurable quantity. Thus any concept of allowable stress in a structural member at one or more critical cross-sections is bound to be semi-empirical.

Dead loads can be estimated accurately; live, wind, earthquake and impact loads cannot even be predicted accurately because of their probailistic nature. However suitable overload factors can be assigned; low values for dead load and high values for others.

The need for accurate determination of loads that may come on the structure is obvious. In the working stress method of design, the total load coming on the structure is considered important and accurate estimates of the individual types of loads is not considered to be essential.

Steel used as reinforcement is manufactured under highly controlled conditions. Hence the properties of the steel can be predicted accurately and can also be controlled at the source by suitable specifications. Concrete, one of the most versatile of building materials and one which is infinitely complicated in its behaviour is not given very much attention, although constant endeavour is made to improve the same. If the design of concrete structures is to be based on certain allowable stresses, it is obvious that concrete should be manufactured under more severe restrictions than perhaps steel. Besides the specified cylinder strength that is to be obtained, it is necessary to control the manufacture of concrete so that uniformity is ensured. This can be done by specifying the mean strength and standard deviation or the coefficient of variation. It is erroneous to specify mix proportions such as 1 : 2 : 4, 1 : 1½ : 3 etc. and then tabulate the allowable stresses for such mixes. The mix design for a stipulated mean strength and standard deviation must be left to the concrete technology expert. This is possible only when the manufacture of concrete is done by specialists,

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attached to a few manufacturing units. Unless very large quantities are involved, the manufacture of concrete at site should be avoided. Even when concrete is manufactured under severe control, there is no guarantee that concrete will actually be placed and compacted properly in the structure. Hence a factor for understrength must be included.

In the past, structures have been build for assumedly infinite, but in fact only unspecified operational life. Recent concepts of the safety of structures involve the thinking in terms of a limited life-span of a structure and a definite probability of failure. We cannot afford to treat an important bridge and a semi-permanent building as having the same life-span. Working stress concepts have been more or less neglecting this aspect of the problem. It is necessary to introduce a set of numbers for the anticipated life-span and the probability of failure for various types of structures. Adjusting the allowable stresses is not the logical approach Statistical and probability theories are highly developed and these should be used in conjunction with load factors or incorporated with the load factors themselves.

The collapse of a column in a multi-storeyed building is bound to be serious. The failure of a beam or a slab may have only local adverse effects. Thus the extent of damage to life and property must also be considered as a factor in the concept of safety.

The behaviour of a beam in flexure and that of a column under eccentric loads is well-known, whereas the shear strength and resistance to torsion are as yet unsolved problems. The safety factor should then include the nature of loading.

Finally, there are structures where the effect of shrinkage and creep are very important. Shrinkage cracks are known to have considerably shortened the life-span of concrete structures. Creep effects are known to increase strains and deflections three-fold in a matter of two or three years.

In the light of all the above, specifying certain allowable stresses for steel and concrete is not a rational concept, more so when stresses cannot be measured and the extent of errors involved cannot be ascertained. Further, the allowable stress concept involving a high factor of safety indirectly implies possibility of bad workmanship, inaccuracies in analysis, and design and encouragement to inexperienced field staff.

If the reinforced concrete construction industry in India is to develop in the right direction, the allowable stress concept should be replaced by methods based on accurate design, supplemented by experimental studies and supported by an efficient construction industry.

3. TESTS ON MATERIALS, STRUCTURAL COMPONENTS AND STRUCTURES :

The properties of steel and concrete have been measured in the past in terms of the yield stress for steel in tension (fy) and the cylinder strength (fc') in compression. Is it logical to specify a single number (fy or fc') as representative of the entire behaviour of a particular structural material? Particularly when load factor concepts are introduced the entire stress-strain curve of the material used in construction becomes necessary so that collapse loads can be computed.

Structural members are rarely in uniaxial compression or tension. There is usually complicated interaction between tensile, compressive, shear and bond stress and the whole problem is three dimensional in nature. However, codes of practice have been giving allowable stresses in terms of the cylinder strength for tension, shea and bond. There is some research evidence th these are functions of; fc' but complete agreeme on this subject has not been reached. The streng of concrete under combined stresses is not y clearly defined. Under these circumstances, must be realised that we are designing structur using a set of arbitrarily chosen numbers such the allowable stresses in concrete and steel. Th is not a rational approach.

Laboratory tests on samples of concrete a steel provide the necessary estimate of the inhere strength of these materials. In the structural con ponents which are made of these materials t same strength need not be expected. It is cle that correlation studies must be undertaken compare the strength of test samples with t strength of actual structural components. In t past this has been taken for granted. The sev rity of loading conditions and the complicate stresses that come on the modern concrete stru ture require the above said correlation.

Load tests on full scale structural component such as beams, floor slabs, columns, frames et are necessary. Tests on these elements upto failu must be made and the behaviour of these membe

should be recorded as load-deflecton, load-strain, moment-curvature and torque-twist curves. The development of crack patterns should be recorded. It is desirable and rational that the performance of the structural elements should be evaluated and future designs based on these test results. Till recently we had only an ' optimistic guess' on these behaviours and only in the last decade, actual tests on structural components have been undertaken.

The study of structural components such as beams columns etc., separately, is just another milestone in the quest for our knowledge of the overall behaviour of the three dimensional structures themselves. Tests on actual structures will be the next step and this has already been attempted on bridges and dams and a few multi-storeyed buildings. Measuring instruments have been incorporated in these structures during construction and they continue to give valuable data. In one extreme case of a test to destruction of an R. C. framed building, the measured collapse load was nine times the computed collapse load of the components, used in that structure.

4. ANALYSIS OF STRUCTURES.

Analysis means the evaluation of moments, shears and thrusts existing at critical cross-sections of any structure, be it simple beams, continuous frames, plates, shells, etc.,

When stresses are small and when concrete has not cracked, the elastic methods of analysis provide reasonable answers. Elastic methods of analysis have been in use for the past several centuries and there are about thrity different methods such as slope - deflection, momentdistribution matrix methods energy methods, etc In fact one has a wide choice. Elastic methods of analysis are regorous. While the present trends in design are not favourable to allow able stress concepts, the author fully appreciates the rationality, versatility and efficiency of elastic methods of analysis.

However, when concrete has cracked, these methods cannot be depended upon. Elasto-plastic methods of analysis will provide reasonable results. The methods proposed by Guyon and Ernst and those proposed on the basis of complementary energy methods are the only usable theories. Elasto plastic methods have be advantage that they can be extended to the elastic and collapse stages. How ever, they are quite cumbersome to use as they exist today. Further research work is needed to simplify the existing methods. Concrete structures, which crack even underservice loads, can be analysed rationally only on the basis of elasto-plastic methods.

When the structure is at the instant of collapse, plastic-hinge theories and the yield-line theory for slabs provide a realistic approach. However, the plastic-hinge theories as developed for steel structures do not seem to be satisfactory for concrete structures. Concrete, when cracked, provides discountinuity in performance and test results do not seem to agree with the theoretical predictions. Fur ther, even if the collapse loads can be estimated accurately, a check for satisfactory performance under service loads becomes necessary. This again emphasizes the need for elasto-plastic theories. The proposal made by Valerie petchu, the optimum redistribution principle, seems to be complicated even for continuous beams.

Elastic theories have been valuable for the analysis of prestressed concrete structures. In this case cracking has been prevented under service loads and hence the usefulness of the concept. For reinforced concrete structures even under small loads, concrete cracks and unless arbitrary adjustments are made for the evaluation of EI, elastic theories are not applicable. Thus the problem we face is the cracking of concrete. If only the tensile strength of concrete can be increased to double its present value, the cracking under service loads can be delayed and the highly developed elastic methods of analysis can be used.

However, the use of elasto-plastic theories cannot be delayed for long. Improvements in the use of this methods are urgently required. An extensive research programme should be initiated in this direction all over India.

5. DESIGN OF STRUCTURES :

Design is the art of providing suitable resistance at all points of a structure so that the effects of loads are resisted at the service load conditions without showing signs of distress (excessive deflections and/or cracking) and to have sufficient capacity to take overload and to counteract unforeseen effects. In the entire field of structural engineering, the design of structures still remains an art; and it is in this area the ultimate strength concepts have the maximum impact.

(1) It provides an accurate estimate of the ultimate flexural capacity of a cross-section. In

simple determinate structures, in conjunction with proper load factors, the exact provision of required flexural resistance can be achieved. One of the major unknowns in the working stress method is thus removed.

(2) Overload factors for dead, live and impact loads can be varied suitably.

(3) Conventional working stress concept for columns is based on ultimate strength and with the introduction of the ultimate strength design concept, this inconsistency in concept can be rationalised.

(4) A better evaluation of the critical moment-thrust ratio for members subject to combined bending and axial load is obtained by ultimate strength design procedure.

(5) It can be shown by ultimate strength tests the spirals do not contribute to additional load carrying capacity of columns, but they serve to increase the toughness of the columns. Both spirally reinforced and tied columns have the same ultimate load capacity.

(6) Ultimate strength concepts show that compression reinforcements are not needed where they are demanded by the working stress concept. Often they serve no purpose and it is the tensile steel that should have been increased.

(7) Yield-line theories for slabs of complicated shapes and boundary conditions are easier for computations than the corresponding elastic theories, even if they are available.

(8) Ultimate strength concepts have satisfactorily explained the effects of settlement, shrinkage and creep on continuous structures.

(9) Ultimate strength tests on continuous, structures have shown the way to take advantage of redistribution of moments and shears.

(10) Ultimate strength theory is founded upon proven behaviour of reinforced concrete members in numerous laboratory investigations.

However, if ultimate strength theories are used, attention must be paid to deflections and cracking under sevice loads. Further, ultimate strength in shear, bond and torsion are as yet unsolved problems.

It is the ultimate strength concepts which elevate the existing design methods from an art into a science. It can be even said that it is the ultimate strength concepts which enable us to have some insight into the fundamental behaviour of concrete structures. Previously we have been having a few 'optimistic guesses' regarding the behaviour of structures.

6. CODES OF PRACTICE :

Since the design of structures is still an art, for the same structure several alternative designs can be proposed. Codes of practice of various countries attempt to narrow down the differences at least in a few important aspects. The code of practice is not a legal document and it is expected, at its best form, to reflect the current thinking in the art of design. Many of the provisions in the codes are restrictive in nature so as to prevent costly errors being commited by the average engineer. Sophistication in analysis and construction often result in unrealistic proposals, since minimum thickness, minimum temperature and shrinkage reinforcement and minimum deflections as specified in codes control the problem.

A critical examination of a few provisions in the existing code IS : 456–64, will be made so as to indicate their arbitrary nature and show that the expert and the research worker need not always be restricted by these provisions.

(1) Concrete has been arbitarily graded into mixes such as M 100, M 150, ...M 400 and allowable stresses are based on these proportions only. The need for specifying grades of concrete is not clear. As in the ACI code, the allowable stresses, if they are still required, can be given as a ratio of the cylinder strength. The grading of concrete into one of the seven mixes allowed in the code is unnecessary.

(2) Axially loaded elastic column design is based on ultimate strength concepts while the beam design is based on elastic theory-there is an inconsistency in the approach.

(3) The spiral is supposed to contribute to the load carrying capacity of the columns. The spirals, according to experiments conducted thirty years back, contribute to the toughness of the column only and hence this provision is misleading.

(4) Column designs under combined bending and axial loads, as done by the elastic theory,

must be checked for ultimate load capacity as per the present code. This provision could have been extended to beams also.

(5) Arbitrary reduction or increase in computed moments at supports by 15% is permitted to take advantage of moment redistribution occuring at high overloads. In the ACI code only ten percent is allowed. While it is well known that redistribution is of considerable help, the performance of such beams under working load conditions should have been checked by experiments. Such arbitrary provisions in codes or practice are superfluous. The expert analyst and the designer will automatically take note of such possibilities and in inexperienced hands such arbitrary provisions may lead to unsafe designs.

(6) Ultimate strength methods of design for beams and columns have been introduced in an incomplete form as compared with the ACI code. The code must give facilities for the designer to use ultimate strength methods of design for columns, beams in flexure and shear and for torsion. The introduction of ultimate strength design could have been delayed until quality control in concrete had been assured.

(7) There are three methods of design for two-way reinforced slabs. The code of practice should be able to specify as to which of these is the best. The other methods could have been omitted.

7. CONCLUSION :

In conclusion the following philosophical concepts can be considered to be representative of the present day approach in the field of reinforced and prestressed concrete construction.

(1) Any procedure which is rational, realistic and economical will be acceptable to the engineers and the rest should be eventually rejected.

(2) We must learn to think in terms of loads and not in terms of stresses.

(3) Specifying certain allowable stresses for steel and concrete is an attempt at over simplification. Concrete design should involve overload factors, understrength factors, probability of failure and quality control on concrete.

(4) Tests upto destruction on samples of materials used, components of structures and full scale tests on structures themselves should form the basis for design. Measuring instrument must be incorporated in the structures themselves as is the practice in dams and actual behaviour of structures must be recorded.

(5) Elastic methods of structural analysis can be used until such time when elasto-plastic-methods appear in a simplified form.

(6) The behaviour of structures from Zero load to failure must be known to engineers accurately. Designing, structures under working loads only or for ultimate load conditions, or mixing the concepts where convenient, is not the rational approach. Formulae for the computation of the performance of the structure at working load range, elasto-plastics range and at collapse loads must all be available. It should be left to the designer to choose what is needed.

(7) Codes of practice must be more rational. Arbitrary provisions in codes should be avoided.

MODEL STUDIES FOR CAUVERY REGULATOR

By

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SYNOPSIS

Two dimensional electrical analogy experiments have been conducted, to examine the safety of the Cauvery regulator, against uplift, in the proposed improvements to the structure.

One of the proposed improvements suggested, is for providing an extended apron 60.00ft or 100.00ft. long, beyond the existing apron, on the upstream side. Trials have been carried out, for different sections under varying depths of permeable subsoil and the values of percentage pressures have been obtained. The results have shown that for an increased length of apron with a cut-off at the end of the extended apron, there is a considerable reduction in the uplift pressure values, and the objective of the improvements is thereby achieved viz-design of the Cauvery Regulator for a higher differential head of 15.00 ft. This study and the data collected, are reported in this paper.

1.00 INTRODUCTION

1.01 The grand Anicut in the Cauvery system is a hydraulic structure dating back to many centuries. The anicut was conceived and constructed by an early Chola King, and is located at the east end of the Srirangam Island in Tiruchirapalli district. The actual anicut is across Ullar to keep back the waters of the Cauvery from draining into the Coleroon and to secure the depth required in the Cauvery.

1.02 The Grand Anicut originally consisted of a solid mass of rough stone in clay 329.00m (1080-00 feet) in length, 12.2x18.30m (40.00 feet to 60.00 feet) in breadth, and 4.57x5.50m (15.00 to 18.00 feet) in height, across the Ullar Channel. Subsequently, a bridge to carry a roadway over the anicut was provided, in addition to scouring sluices. The structure has been found to have a central core of roughly hewn granite. This has been set in mud and then covered by an outer facing of concrete all in lime mortar. The anicut was not of uniform shape or section. Later, improvements were effected to the anicut for stabilising the water supply to the Thanjavur delta region, further down the anicut. The Grand Anicut was subsequently remodelled as a bridge cum regulator with lift shutters and has been functioning ever since as an anicut and a surplus work, immediately below which the main river cauvery branches off into Cauvery and Vennar.

1.03 The problem of the Cauvery bed getting silted up, gradually, arose in the wake of the construction of the Grand Anicut. This was tackled by Sir Arthur Cotton and the "Upper Anicut" was constructed at the head of Coleroon in 1836-38. This structure was designed to pass adequate discharge into the Cauvery. The works at the upper anicut are (i) a barrage across the Coleroon with lift gates (ii) a bed-dam across Cauvery river to prevent scouring of the bed and (iii) a surplus to serve as a spill channel to take off a part of the surplus water into the Coleroon below the Upper Anicut.

1.04 New sluices were then added at the down stream end of the Grand Anicut as later improvements. A road-dam about 1237.50m (4060 feet) in length constructed on the left bank of the Cauvery about 1½ miles upstream of the Grand Anicut was intended as a surplus work in times of unprecedented floods. The new grand anicut canal head sluice was built on the right bank of the Cauvery opposite to the Grand Anicut and adjoining the Vennar regulators.

The details of the Grand Anicut are indicated in Figure 1.

2.00 NEED FOR IMPROVEMENTS.

2.01 On a recent examination of the Cauvery and Vennar regulators, it was found necessary to strengthen both so that during high floods the regulators could be completely closed down. The site engineers reported that experimental studies were necessary for this strengthening work, wherein the regulators would have to withstand uplift

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pressures for a higher differential head of 15.0 feet, overcoming the limitations of the restricted operations in vogue under present rules viz., the difference between the upstream and downstream water-levels should not exceed 9.00 feet.

The problem was accordingly referred to the Soil mechanics and Research Division, for conducting electrical analogy experiments for finding out the distribution of uplift pressures below the regulator floor under various conditions, incorporating the proposed improvements.

3.00 MODEL TESTS:

3.01 A section of the Cauvery Regulator was furnished by the site Engineer for adoption for the model studies, for considering its safety against uplift pressures. The structure being an old one, the depth of the permeable sub-soil below the regulator floor was taken as infinite or indeterminate. Trials were also conducted for certain assumed finite depth values for the pervious sub stratum.

3.02 The first trial in the study was for a section with a cut-off at 40.00 feet downstream of the shutters as per design requirements. The profile of the bottom of the regulator was reproduced to a scale, 1 cm = 5 feet in the two dimensional electrical analogy tray. The pressure distribution below the floor was traced by 5% equipotential lines.

3.03 The experimental results indicated that the entire effect of the uplift pressure was felt on the upstream apron, upto the 50% equipotential line. The thickness of the existing apron, upstream of the main regulator was observed to be not sufficient for a differential head of 15.00 feet.

4.00 FURTHER STUDIES.

4.01 Further studies were taken up for some more sections for observing the change in the pattern of uplift distribution under varying conditions. The upstream apron was extended by a length of 60.00 feet in front of the existing apron. This was tried as a means of increasing the path of percolation for the sub soil water to reduce the uplift pressures that would be acting at those points. Two trials were conduucted for depths of infinite substratum and for a depth equal to the basewidth of the regulator floor (T = b)

4.02 Studies were also conducted with a cut-off at the upstream end of the extended apron. This was in addition to the existing sheet pile cut off. Trials were conducted for four different depths of permeable sub soil below the upstream bed (T = Infinity, 1.0b, 0.5b and 0.2b respectively).

4.03 The section of the regulator as given by the site Engineers and as adopted for the model studies is given as fig. 2.

4.04 The two dimensional electrical analogy tray was used for tracing the equipotential lines. Potentiograms in steps of 5% were traced. The actual uplift pressure values at significant points were calculated.

4.05 The uplift pressure values at various points for the different trails are furnished in figure 3. In figure 4 the fall of uplift pressure along the bottom face of the floor, for the different conditions has been plotted for ready reference and comparative study.

4.06 The actual values at the critical points viz., first in front of the regulator foundation, at C and D, the pressure drop across the existing pile line A & B and other points have been calculated.

5.00 DISCUSSION OF RESULTS

5.01 The pressure values at C and D are furnished below for the three different conditions.

TABLE I

Condition.	Pressure % at C	D
1. Existing Section.	50.0	45.0
2. Existing section with extended 60 feet apron without cutoff.	45.8	42.5
3. Existing section with extended 60 feet apron with cut-off.	42.2	41.0

Both at C and D there is a considerable reduction in uplift pressure by providing the extended apron and the cut off. The pressure drop cross the pile line (C & D) seems to be much less, with the provision of the cut-off at the end of the extended apron. The drop in percentage pressures across AB is from 10% to 7.2% vide Table 2.

5.02 The pressure values tend to increase with decreasing depths of permeable sub-stratum, confirming the general observations under similar studies. The magnitude of variation, as seen from the values at A and B (Table No. 2) is nearly 79.% and 12.5% respectively, for a value of T = 1.0b for the permeable stratum. In the case of the extended apron with a cut off at the upstream end, the value of pressure at C is not much affected by a finite depth (T = 1.0b) of permeable sub soil. This is directly the influence of the new cut-off which changes the pattern of distribution. At 'B' (downstream side of the existing pile line) the pressure is 66.4% for the original section and 52.8% for the new section with extended apron and cut off. The effect of the extended apron and the cut off are amply brought out by the results.

5.03 In this connection it is pointed out that the field engineers have clearly stated that the structure is an old one with infinite depth of permeable substratum below and that solutions for such a condition only are to be preferred.

6.00 CONCLUSION

6.01 The necessity for an extended apron and the cut off at the front end have been confirmed by the test results, as discussed in the preceding paragraphs. For the safety of the upstream apron, the minimum practicable and economical thickness and the exact length of extension on the upstream side have to be worked out on the basis of design assumptions, as there is always standing water over the upstream apron.

6.02 The electrical analogy studies have given fairly accurate and quick results, incorporating the different variable parameters in the model.

TABLE 2.

Pressure values % for floor with extended 60 feet apron but without cut off.

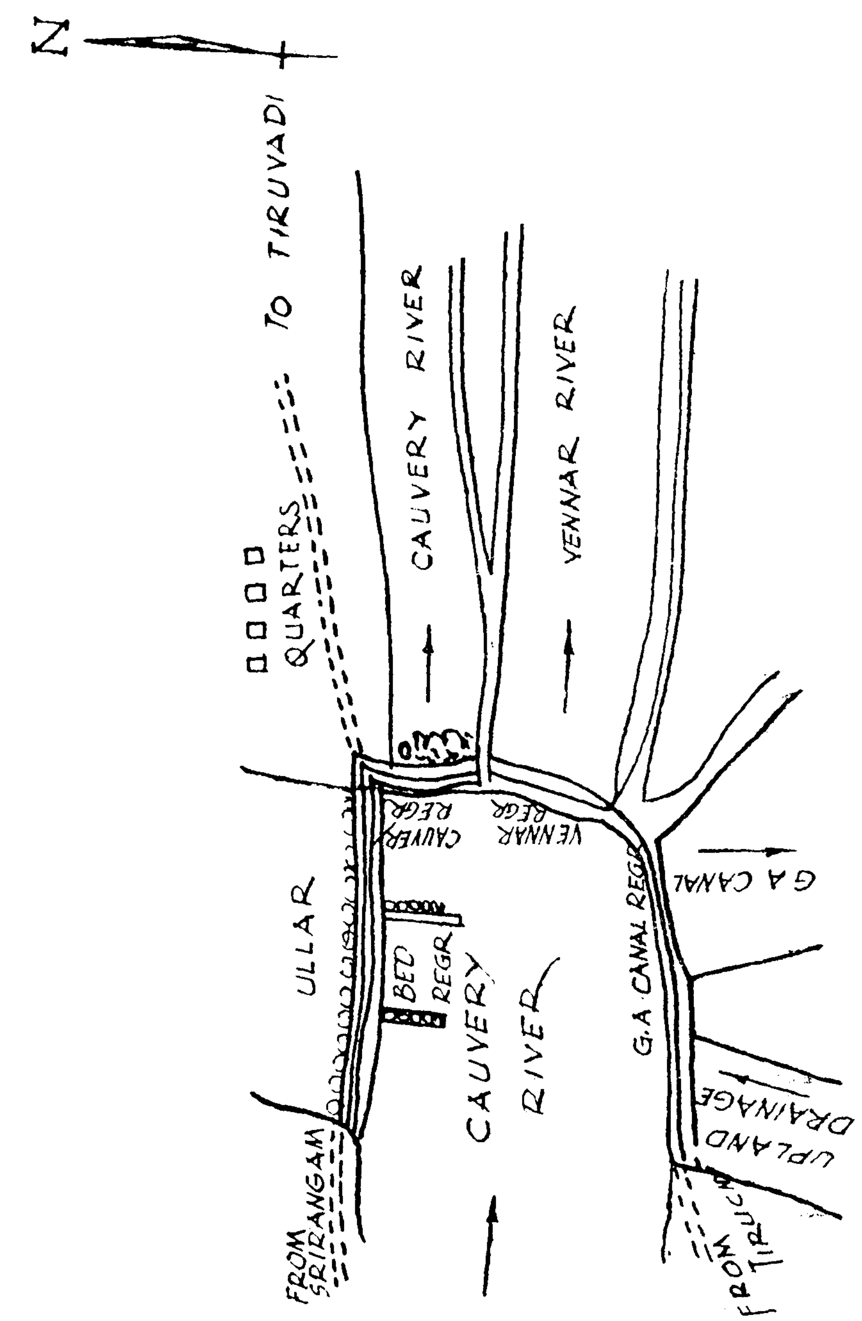
T = depth of stratum.	Pressure % at *A.	B.	C.	D.	E.
Infinity	65.0	55.0	45.8	42.5	32.5
1.0b	72.9	67.5	53.7	48.5	36.8

TABLE 3

Pressure values % for floor with extend 60 feet apron with cut off upto depth + 163.25.

T = depth of stratum	Pressure values % at *A'	B'	A	B	C	D	E
Infinity	100	70.0	60.0	52.8	42.2	41.0	30.0
1.0b	100	70.0	61.5	53.0	43.6	41.3	31.3
0.5b	100	72.1	62.0	53.1	43.6	41.4	31.3
0.25b	100	75.0	65.0	57.0	45.0	43.2	31.5

(*For location of points A', B', A B, C E, D E, please refer to Figure 4.)



NOT TO SCALE

Figure 1.

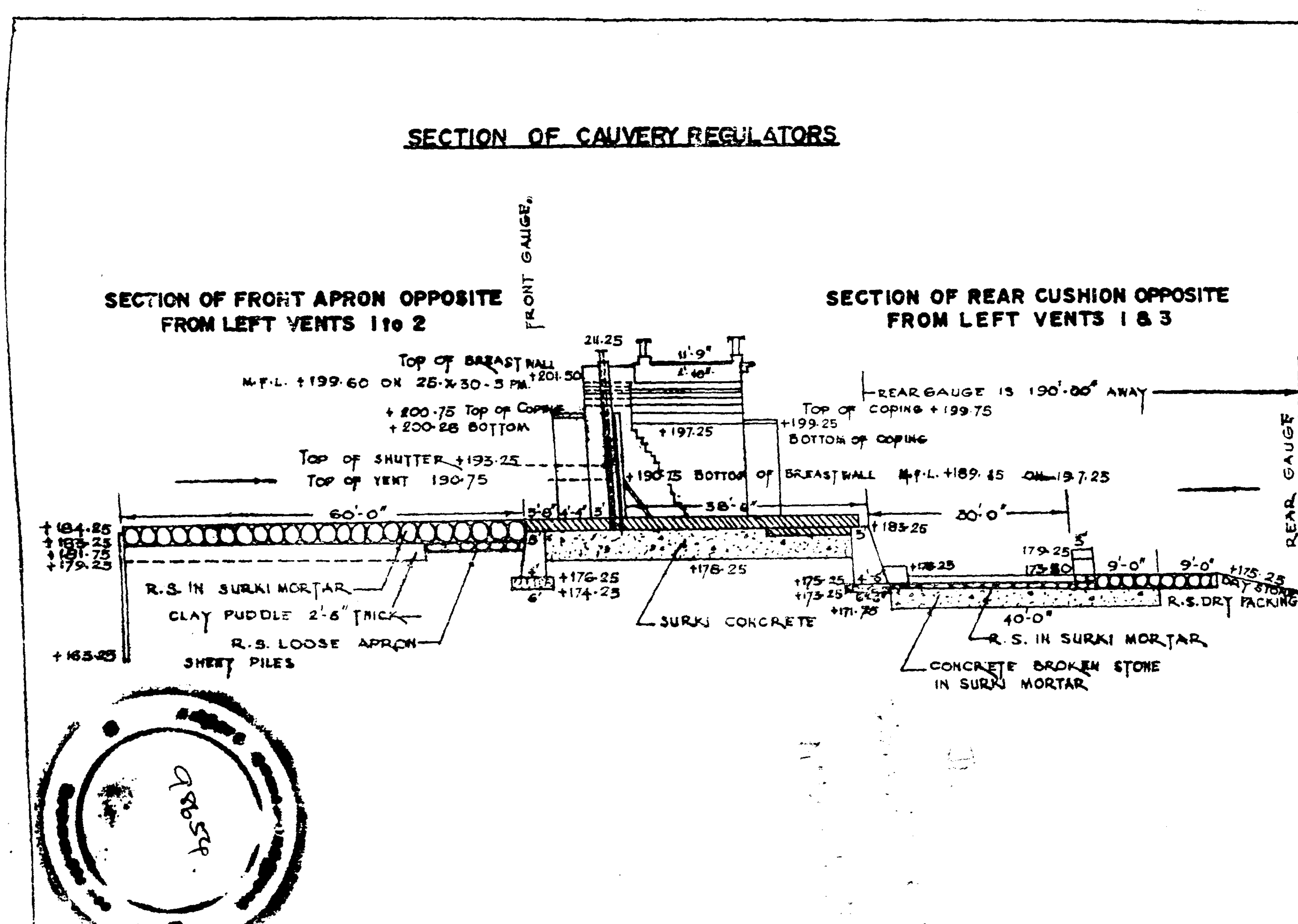

SECTION OF CAUVERY REGULATORS
SECTION OF FRONT APRON OPPOSITE FROM LEFT VENTS 1 to 2
SECTION OF REAR CUSHION OPPOSITE FROM LEFT VENTS 1 & 3
FRONT GAUGE
REAR GAUGE
REAR GAUGE IS 190'-00" AWAY
Top of BREAST WALL
Top of SHUTTER +193.25
Top of VENT 190.75
Top of COPING +199.75
BOTTOM of COPING
BOTTOM of BREAST WALL
60'-0"
38'-6"
30'-0"
40'-0"
9'-0"
R.S. IN SURKI MORTAR
CLAY PUDDLE 2'-6" THICK
R.S. LOOSE APRON
SHEET PILES
SURKI CONCRETE
R.S. IN SURKI MORTAR
CONCRETE BROKEN STONE IN SURKI MORTAR
R.S. DRY PACKING
98654

Figure 2.

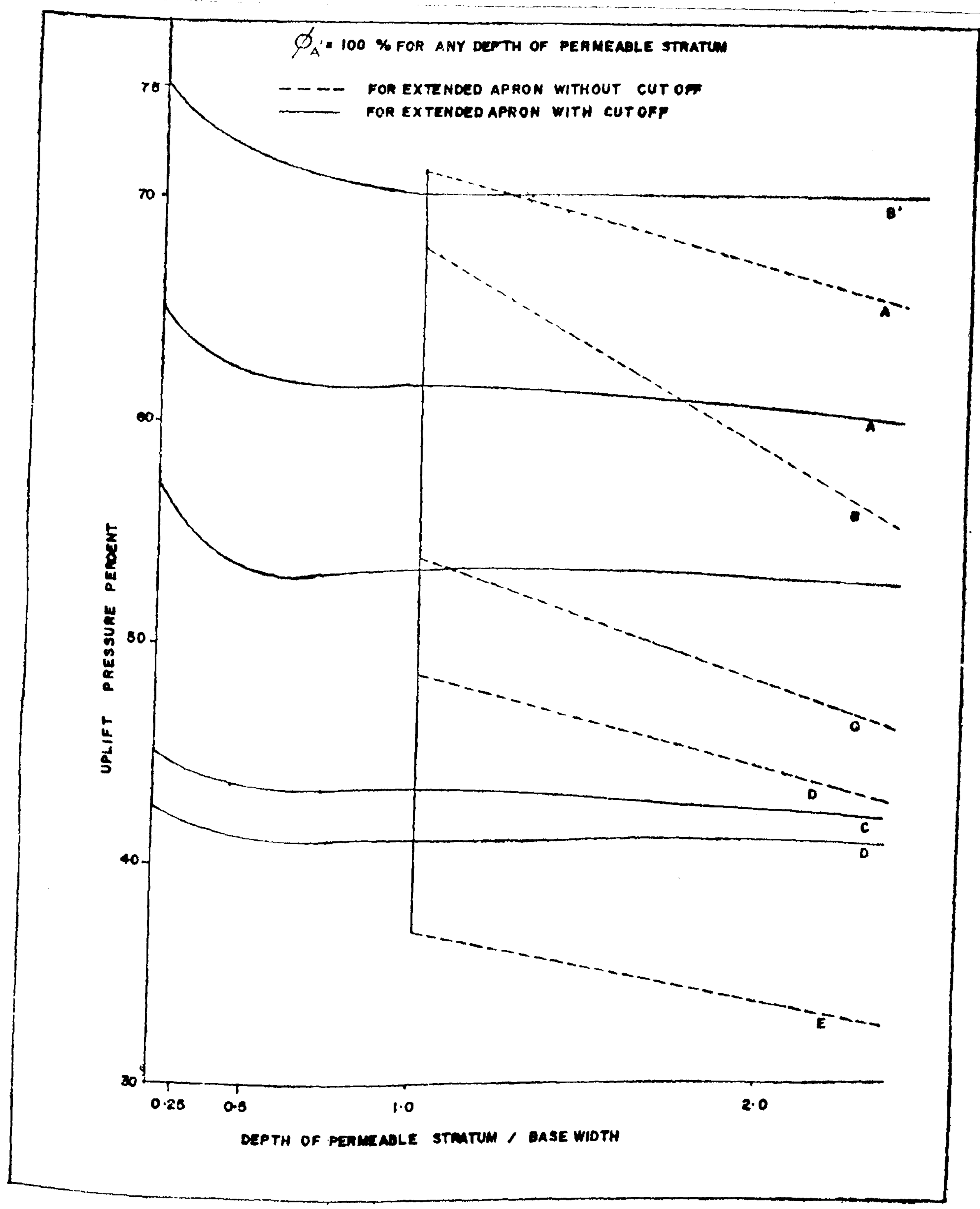


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Figure 3. Uplift variation for different depths of Permeable stratum at different points (Vide Table 2).

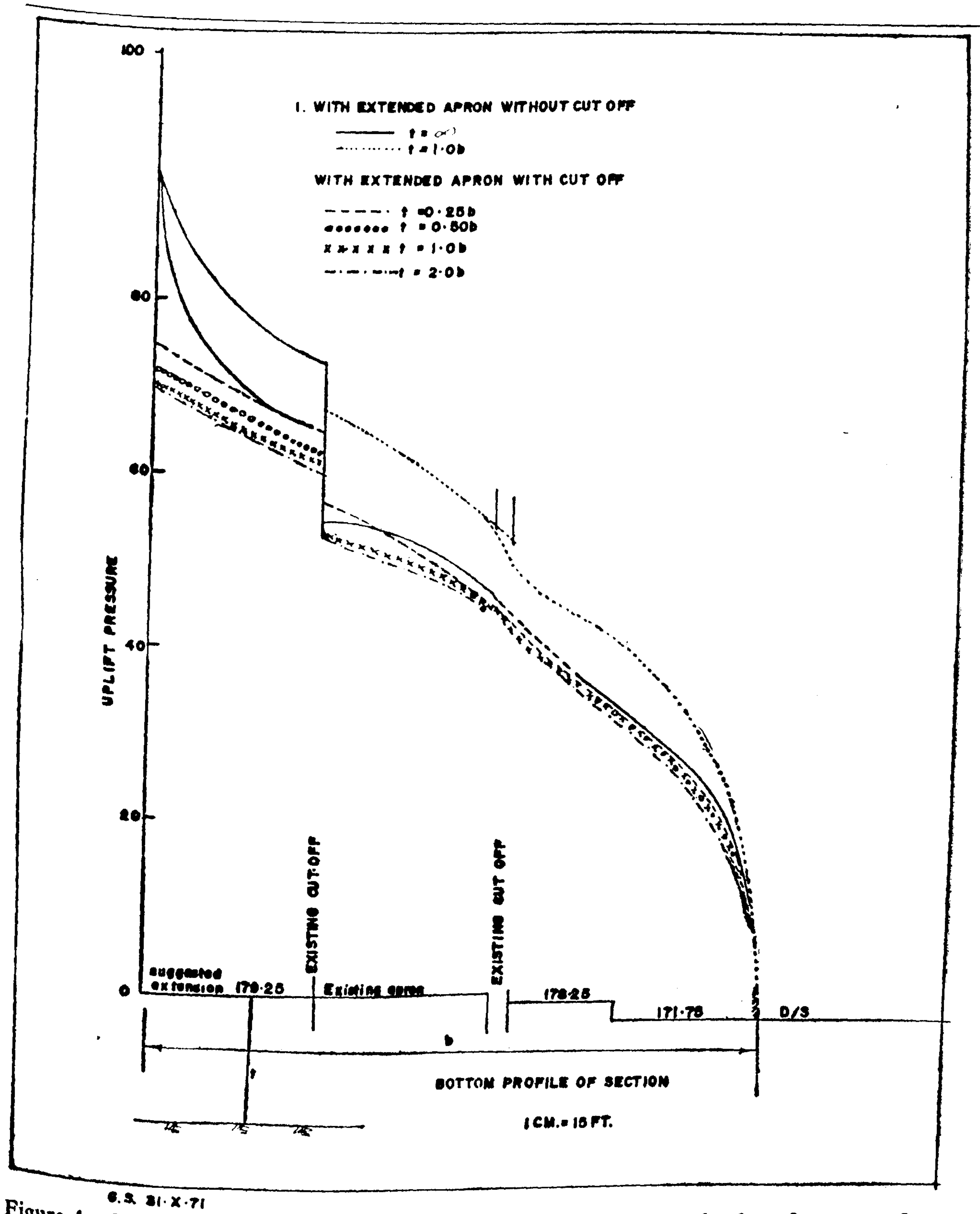


Figure 4. Figure showing fall of uplift along the floor, for different depths of stratum for extended apron with & without cut-off.

27th Zonal Meeting on Research Scheme to River Valley Projects Held in Madras from 28th September to 30th September.

The Soil Mechanics & Research Division, Madras and the Irrigation Research Station, Poondi have been for long associated with the Central Board of Irrigation and power in carrying out Research Investigations on fundamental and basic problems of engineering under grants-in-aid given by the Central Board of Irrigation and Power. To review the work done on the research schemes by these and several other Institutions the Central Board of Irrigation and Power held its South Zone Meeting in Madras from 28th September, to 30th September 1970. The Meeting was hosted by the State of Tamil Nadu. A number of delegates from all over India attended the meeting and the Seminars that followed.

The meeting was inaugurated at the Rajaji Hall by Thiru S. J. Sadiq Pasha, Hon'. Minister for Public Works, Tamil Nadu and presided over by Thiru O. P. Raman, the Hon. Minister for Electricity.

In his inaugural address the works Minister advocated a water policy at the all India level and a master plan for proper use of the water resources. It is only by bold schemes like linking up the Ganga with the rivers of South, food and economic problems could be solved in the long run. He also emphasized the need for taking steps towards economic use of water, conservation of existing resources initiating measures for proper rational distribution and evolving a National Plan for the use of water. As far as Tamil Nadu goes, the Minister observed that the surface water had been fully used and hence attention should be directed to taping of ground water resources.

The Hon. Minister for Electricity in his presidential address dealt with in detail the developments made in regard to power generation during the last two decades. He felt that the Hydropotential in the State being small it was necessary to go in a big way to set up large thermal or atomic power stations to meet the power demands.

In his welcome speech, Thiru J. E. Vaz, Chief Engineer, General & Irrigation, Tamil Nadu stressed the important role that research Engineers will have to play in future in the development of water management practices. He also said that Tamil Nadu had made considerable progress in the integrated utilization of surface and ground water.

Thiru S. K. Jain, President of the Central Board of Irrigation and Power who also participated said that the valuable work done by the research personnel engaged in the Central Board of Irrigation and Power Schemes was of considerable assistance to the Engineers executing the engineering projects.

Thiru J. Walter, Superintending Engineer, Designs and Marine Works Circle proposed a vote of thanks.

After the inaugural function, the Technical Sessions on soils and concrete were held on 28th September 1970. These were followed by a Seminar on Hydraulics on 29th and 30th September 1970. The research aspects under the scheme which were not covered by the Seminar were discussed in the Hydraulics Session after the Seminar. The Session on the 30th was held at the Irrigation Research Station, Poondi.

In the Technical Session on soils, problems relating to Engineering properties of soils vibrations in soils and concrete and compaction of soils in standing water with particular reference to construction of earth dams were discussed. A number of cases of failure of the instrumentation installed for measurement of pore pressures and settlement in dams were reported by the various research stations. The possible causes of such failure were reviewed and it was the consensus of opinion that there was need for going into the question in greater detail in the light of experience of the various laboratories in this regard.

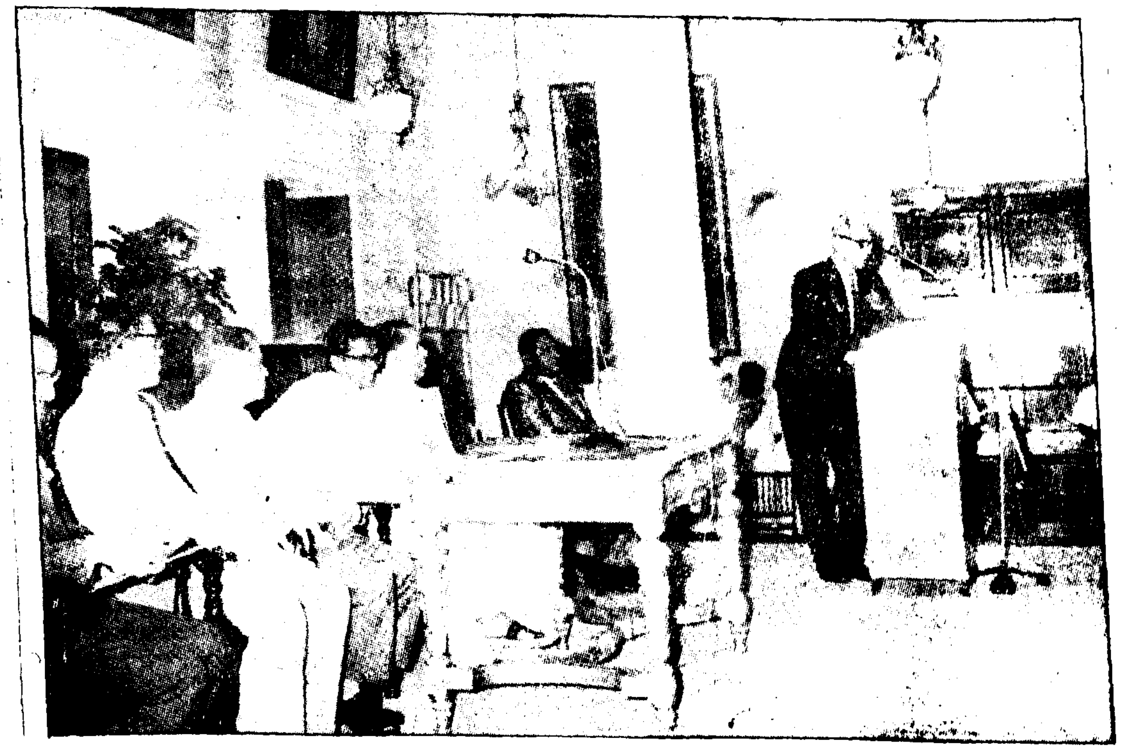
In the Technical Session on Concrete, problems: " Use of Surkhi and other pozzolanic materials in mortar and concrete, " " Principles of mortar and concrete mix design ", " Precast techniques in Hydraulic structures ", " Selection, processing and specifications of aggregates for use in concrete in hydraulic structures " were discussed. The question of using flyash in reinforced concrete evoked considerable discussion. Opinions were expressed on the basis of the researches carried out that there was no increased hazard of corrosion of the reinforcement in concrete as a result of use of flyash in concrete, as long as flyash conforming the standards is used and it is possible to make a dense and impermeable concrete.

Six working papers prepard by Central Water and Power Research Station, Poondi ; Andhra

Pradesh Engineering Research Laboratory, Hyderabad, Soil Mechanics & Research Division, Madras, Mysore Engineering Research Station, Krishnarajasagar and Indian Institute of Science, Bangalore covering the aspects, " Factors affecting the design of alluvial channels, sedimentation of reservoirs, design of hydraulic structures on permeable foundation, canal linings, effect of air entrainment on the optimum sizes of stilling pools and other energy dissipation arrangements and cavitation in hydraulic structures and methods of prevention were presented. Lively discussion took place on all these aspects.

During discussion on the factors affecting stability of alluvial channels and methods for their design under controlled discharge, sediment grade and charge etc., it was felt that only the more significant factors responsible for channel stability need be considered and a study made to evaluate their effect on the design of stable alluvial channel. The Lacey's regime concept and the tractive force approach to the design of alluvial channels were reviewed in the light of Lacey's own modifications to his earlier suggested equations and the recent researches carried out by various investigators in this regard. The consensus of opinion was that the tractive force approach was not yet fully perfected and as such resort will continue to be made to the empirical regime theory. It was felt that considerable alteration had taken place in Lacey's regime theory and so a reappraisal of this theory was required to be made to include the latest procedures to be used in the design of alluvial channels. The importance of collecting extensive data on stable prototype channels was emphasised to achieve this end.

The discussion on the important problems of sedimentation of reservoirs centred on the difficulties encountered in the conduct of sedimentation surveys. Particular mention was made of the difficulties faced in fixing the range monuments after the reservoir came into operation. It was suggested that the work of fixing the range monuments should be carried out before the first impounding of the reservoir. The use of electro and sonic instruments for measurement, along ranges and for triangulation survey was suggest The need for conducting repeat sedimentat surveys in Tungabhatra and Lower Bhavani reservoirs was stressed with a view to ascertain whether sedimentation continued in these reservoirs at the rates indicated by the earlier surveys of decreased with the age of reservoir. It was reported that soil conservation studies had been initiated in a few reservoir catchments in Tamil Nadu and Maharashtra. Similar work is required to be



Thiru J. E. Vaz, Chief Engineer, General & Irrigation, delivering the welcome address

taken up in catchments of reservoirs in other states which have shown a high rate of silting and the effect of these measures in reducing sedimentation in erservoirs should be reported.

The working paper proposed by Thiruvalargal J. Walter, L. Venkatanarasimhan, M. N. Raghuraman, and T. S. Krishnamurthy on the " Design of Hydraulic structures on permeable foundation was discussed in detail. It was brought out that it was only in India and the USSR that such extensive studies have been carried out on this important subject.

The working paper on 'Canal Linings' evoked considerable discussion in regard to the hydraulic and structural efficacy and economics of various types of canal linings. A number of additional factors such as small cost of cross masonry works which had an influence on the economics of canal linings were stressed. The need for treating distributaries and minors on a different footing was also mentioned. With reference to the use of asphaltic linings a note of a caution was sounded, as it was found that though the lining was satifsactory in the initial stages, the lining deteriorated very rapidly after a few years.

The Indian Institute of Science, Bangalore in its paper on the effect of air entrainment on the optimum sizes of stilling pools dealt with the design of a pre-entrained hydraulic jump stilling basin. Aspects such as inception of air entrainment, mean air concentration of uniformly aerated flow, energy content at spillway toe, jump characteristics, viz., height and length and energy dissipation in the jump were discussed. It was felt that utility of criteria suggested would be enhanced if a few worked out examples were appended. It was also suggested that the limitation of the criterria be evaluated after carrying out studies on high head prototype structures.

Another working paper presented by the Indian Institute of Science, Bangalore was on the subject of cavitation in hydraulic structures and methods of its prevention. The situations where cavitation is known to have occured were discussed. The limitations of an open air hydraulic model vis-a-vis a cavitation tank in the measurement of negative pressures and consequent prediction of cavitation were also discussed. It was suggested that actual cases of cavitation occurrence in hydraulic structures in the country should be brought to the notice of the researchers so that causes of their occurrence could be evaluated and the knowledge of the occurrence of cavitation phenomenon in hydraulic structures could be enriched.

NEWS FROM ABROAD

(1) RIVER BANK PROTECTION

The problem of protecting the surfaces of water retaining embankments, channels, canals, river and coast protection works against erosion both above and below the water line is a constant concern of river authorities. The costs involved have long made apparent the need for a cheap, simple and flexible system for solving the problem.

To this end, Cementation Ground Engineering Ltd some 18 months Ago, introduced to the U. K. a patented process, based on the injection of a colloidal grout into a flexible form work of two layers of synthetic fabric, known as ULO Mats. After various trials, research and development, Cementation has now used it on a number of sites and work is currently being carried out on the River Arun for Sussex River authority.

For many years protection of the river face of the embankments of the River Arun has been affected by the use of chalk revetment and, in particular, the placing of hand pitched blocks. However, a rapid deterioration of the original blockwork over the 7 miles of river between Arundel and the sea was apparent by 1967 and there seems little doubt that erosion is accelerated by wave action from powerboats which use this section of the river in ever increasing numbers. In 1968 the Sussex River, Authority decided to embark on the first stage of a scheme to protect the channel banks between Little hampton and Arundel at a then estimated cost of £3,00,000

The ULO Mat consists of two layers of prefabricated, high strength nylon incorporating nylon thread spacers which are stitched into the upper and lower layers of the fabric.

At Arun, the River Authority's own labour force, working ahead of cementation, are grading the river banks. The 100mm thick mats used are manufactured in 12m square panels, vertical joints are sewn together as work proceeds.

Each panel is laid out on the ground in the site storage area and checked for faults. Plastic injection pipes, 40m long and 38mm dia are then carefully inserted in the mats at appropriate spacing. The mat is then rolled up and carried to the river bank where it is laid at low tide next to the mat already in position. Special seams which are provided in the mats are then brought together and stitched by an electrically operated sewing machine. This procedure is repeated for two or three panels according to the expected output for the day and there is always one more mat laid down than can be filled to enable the subsequent joining with the next mat before filling commences. The mats are pulled out to their full length and width, the edge beam shutter fixed at the top of the bank and the upper layer of fabric hooked on to the shutter. The lower layer of fabric is pegged to the ground.

A grout, consisting of a 3 : 1 sand/cement ratio is prepared in a colloidal grout mixer and pumped along the river bank and then into the mats. Weep holes are formed after the grout has set to relieve any water pressure behind the ULO Mat. Output has reached 350m²/day.

Judging by the reception that the River Authority has given the 1400m of river bank contracted to Cementation they are well placed for a further section.

(The Consulting Engineer January 197.)

(2) NEW SLUDGE SYSTEM CUTS DISPOSAL COSTS

Most effluent treatment plants produce sludge which must be either disposed of by tanker or dewatered and dumped as a cake. Neither of these operations is cheap. Serck Eftrol, of Gloucester Trading Estate, Hucclecote, Gloucester, GL3 FAA, claim that their automatic de-sludging system will cut sludge removal costs by half, and repay 200 per cent per annum on its capital cost.

The system gives effective automatic control of the sludge level within certain types of sedimentation vessel, thereby yielding several benefits. First, by increasing the solids concentration, it drastically cuts the volume of sludge which must be removed or dried. Second, it significantly reduces the manpower otherwise needed to effect periodic sludge bleed-off Third, it percludes risks of solids escape due to the presence of excess sludge. Moreover, it reports and alarms, where necessary, any undesirable conditions within the settling tank, giving the operator a precise control over this part of the plant.

The method of operation involves detection of the sludge level by means of a robust device which is sensitive to buoyancy differentials and can be preset to fit the individual case. This device controls valves or pumps to implement the precise amount of bleed-off needed to maintain the optimum range of sludge level. Thus, unlike techniques which depend on timing mechanisms, the Serck Eftrol system adjusts to different effluent flow rates and solids concentrations.

Automatic de-sludging can be installed as part of an overall Serck Eftrol effluent plant, or can be supplied for incorporation into existing plants of other manufacture. The system is also applicable to other industrial process involving sedimentation. (The Architect and Building News : Sept. 3, 1970)

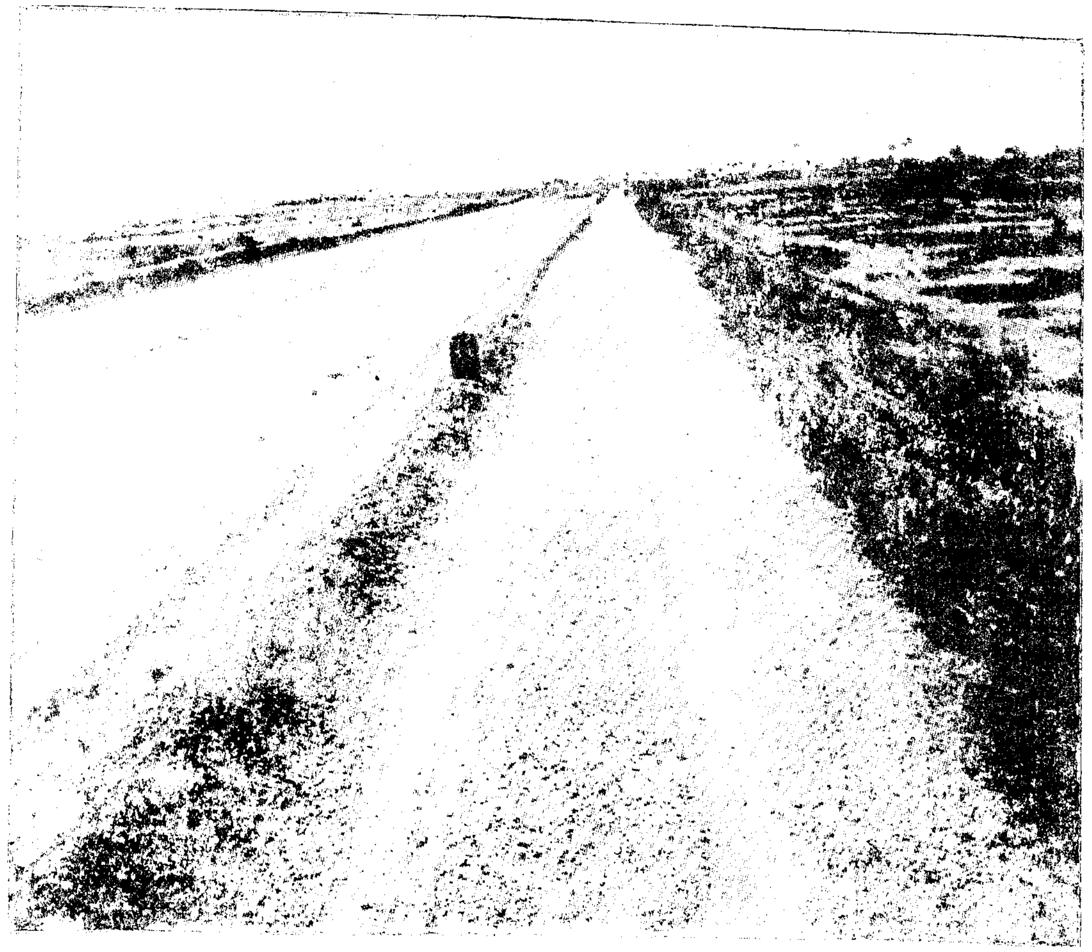
(3) RETAINING WALL FOR BASEMENT CAR PARK

The Construction of a cast in situ 500 ft. long retaining wall for a basement car park, forming part of extensions to the existing four-storey Post Office Telephone Exchange at Uxbridge.

The retaining wall, designed as a diaphragm wall and constructed using the bentonite displacement technique, was 20 in thick. The depth of diaphragm walling varied from 15ft. in the ramp area to 28ft. in the main basement area, the maximum exposed height of wall being 21ft. Starter bars were incorporated in the steel cages behind polystyrene formers, the bars being subsequently bent out to form a key to a nib in the floor slab.

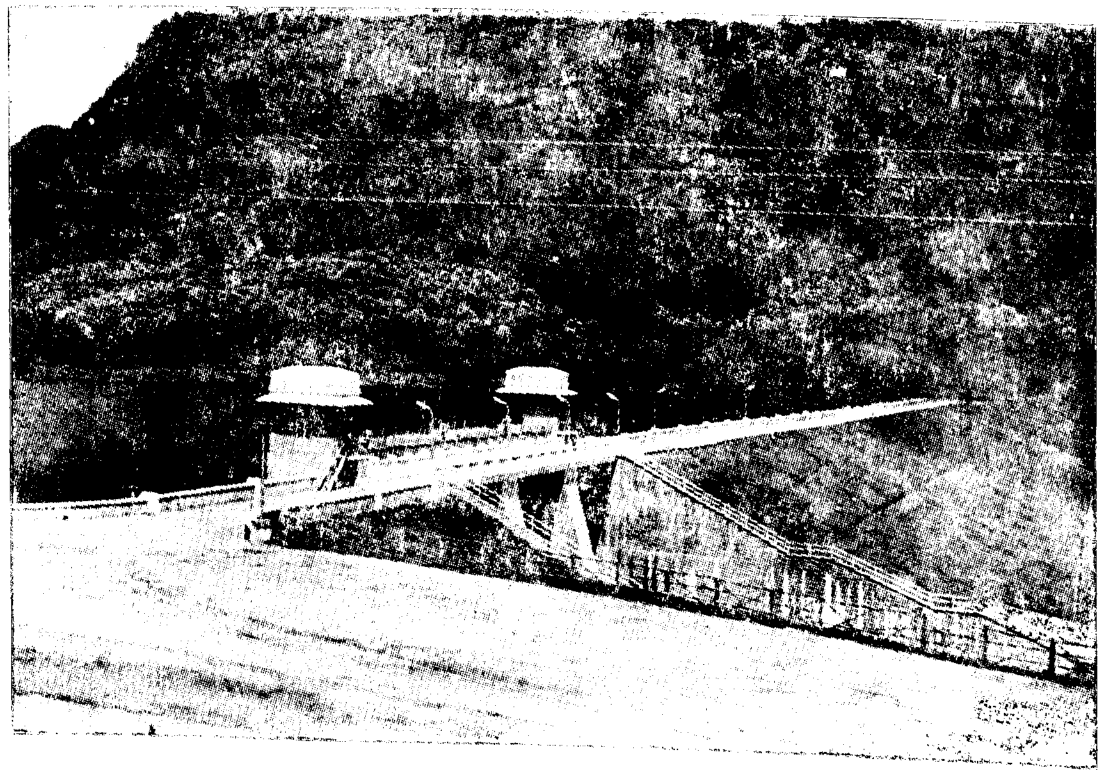
To reduce the excavation period to a minimum, a triple type support system for the diaphragm wall was installed. This involved normal tie-back anchorages installed by a BBE 57 rockdrill and consisting of 20mm Macalloy bars up to 62 ft. long, grouted into the soil; foundation bolts of the duplex type to tie the diaphragm wall to the base slab foundation of the existing Telephone Exchange, where it was not possible to use tieback anchors, due to the proximity of existing services and adjacent properties, horizontal and raking shores were used. The vertical component of the load was resisted by means of plates secured to the wall by 1½ in dia bolts, thermic lancing being used for their installation.

(Civil Engineering and Public Works Review November, 1970



A VIEW OF [illegible] MAIN CANAL

A VIEW OF [illegible] DAM



INVITATION TO READERS

Kind attention of the Engineers of the Madras Public Works Department and readers of the journal elsewhere is drawn to the meagre response to repeated requests for contribution of articles to the journal.

May we once again emphasise the two-fold objective of the journal, viz., to promote the interest of engineers and aiding their specialised work by placing before them the many problems of technical importance and ways and means of solving them, and secondly, opportunities being scarce for engineers to express their own experience in solving practical problems of design and execution. We wish to add that this journal has been the only medium of expression for those in the field. It is earnestly desired that the Engineering fraternity will come forward in large numbers to utilise this medium to its best advantage by offering their valuable contribution.

The scope of the articles to the journal need not be rigorously confined to the subjects on irrigation alone. Articles which have a direct bearing on subjects given below are also welcome :—

(1) Technical aspects of foundation soils and foundation Engineering.

(2) Auxiliary works in the field of irrigation and other water retaining structures and works.

(3) Practical solution to hydraulic problems in canals and distribution systems with reference to their maintenance.

(4) Problems of soil erosion and conservation and reclamation of land.

(5) Practical silt problems in rivers and reservoirs.

(6) Problems of sanitation and public health and possibilities of utilisation of sewage.

(7) Inland waterways and navigation, fishing schemes and protection of land against sea erosion, etc.

(8) Problems of drainage in deltaic tracts and entrance of rivers into the sea.

(9) Run-off studies in catchments and evaporation studies in reservoirs.

(10) All allied subjects.

The Executive Engineers of the Madras Public Works Department are specially requested to encourage their Subordinate Officers to come forward with articles and enable the journal to flourish and serve the common cause of the Engineers of the State.

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