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or a novel approach to a problem. Further correspondence from him with you after receipt of the above will be in the form of asking for a few particulars for which also you may send copies of letters existing in files or for some omitted details in plan which means only work for the draftsman. The staff in Gauging Division, Madras may even go over to the field with a photographer in case the situation, so warrants.

With these encouraging assurance offered I hope the flow of articles for this Journal will increase many fold and the purpose for which this journal is published will be fulfilled to a greater degree.

CONCRETE TECHNOLOGY

By

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DEFINITION.

Concrete can be defined as a non-homogenous manufactured stone composed of graded, granular, inert materials held together by the action of cement and water. The inert materials usually consist of gravel or large particles of crushed stone and sand or pulvarised stone. Manufactured aggregates can also be used but natural aggregates bring down the cost.

ESSENTIAL REQUIREMENTS.

Concrete is bistrength material, strong in compression but weak in tension. By burying steel or prestreching high tensile steel wires using the beam ends as shoulders, flexure members or container structures can be produced. Because concrete is made of the locally available materials, it can be moulded to any shape and is quite cheap, it is extensively used as a structural material.

Depending on the use to which the material is put in, it can be expected to have a specific quality. The concrete should be durable (i.e.), should not change in volume when once it is cast and has hardened. In Reinforced Concrete tanks and aquatic structures, concrete should be strong and water-tight. In runways in aerodromes and pavements, concrete should be tough, possess high resistance against wear and have high flexural tensile strength. In reinforced and prestressed concrete flexural members, concrete should be strong in compression.

STAGES OF CONCRETE.

Concrete passes through two stages: the fresh state and the hardened state. Fresh mixes can be thought of to be made of the coarse particles surrounded by the lubricating film (viz.) the mortar. The properties of fresh concrete depend on the thickness and consistency of the lubricating film. In the fresh state, concrete should be workable (i) easy to handle (viz.) the amount of useful internal work necessary to produce full compaction conforming with conditions of placing and compaction should be reasonable. The compactability, mobility and stability during handling and vibration of the mix must be fully ensured. A non-intensional air content of 5 percent entails a loss of 30 percent in strength of concrete and a 10 percent air ends in loss of strength of 55 percent. Hence the concrete should be compacted with air content less than 1–2 percent.

In the hardened state concrete should be sufficiently strong to carry loads or resist applied forces (Table-10). The strength depends on the consistency of the paste, since the aggregates are usually stronger and failure occurs at the interfaces. Most of the other properties like flexural strength, water-tightness, wear resistance, durability, stiffness, creep, shrinkage, etc., vary either directly or indirectly with the strength properties. Hence cube strength at a certain age (say 28 days) becomes a basic requirement for quality testing.

ROLE OF THE MIX DESIGNER.

Necessary workability in the fresh state with sufficient strength in hardened state can be produced by any number of combinations but the customer would like to have a cheap one designed for his job. We can either have a low workable mix with a low cement content and expect to use heavy compacting equipments like surface vibrator with a poker vibrator or the same strength mix can be produced by a liberal provision of cement ending with a good workable mix requiring only moderate compaction. The relative cost, the control and type of work and the effect of various parameters on the end product must be thought of and here comes the designer's skill and responsibility.

WORK INVOLVED IN MIX DESIGN.

In department 1 : 2 : 4 volume batched concrete for R.C.C. work is allowed with water content mostly controlled by the mixer driver by the look and feel of concrete (he cannot be expected to know about strength-water relationship). Even with this control, improvement in quality can be thought of. It takes possibly 20 hours of study to understand the various implications of mix design. It takes about ½ hour to design a mix but the follow up works (viz.) preliminary, casting, testing, modification, confirmatory casting and testing, field mix modification taking into account the free water content or absorption allowance for bone dry aggregate to keep the Water' Cement Ratio as designed, testing to know the variability of control for the site conditions require considerable skill and time. Hence a contract for the first mix design may cost Rs. 800 excluding cost of materials but the subsequent mixes for the same site conditions and materials cost Rs. 300.

NECESSITY FOR MIX DESIGN.

We have not been designing the mix for strength so far. We know richer concrete is stronger and has nearly the same workability. By experience constant workability is aimed at and this requires same water content. With richer concrete, the cement content is more and hence Water/Cement ratio is less, giving a stronger matrix and a stronger concrete. We do not have an exact idea of the actual strength of concrete laid, it may be wide the mark say quite safe or encroached well into the factor of safety region. When stee

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is a well defined structural material there appears no reason how we can allow the equally important structural material, viz., concrete to have any strength. I feel by designing the M 150 alone, on a modest scale 10 percent savings in cement is possible resulting a net savings of 2 percent in cost of civil works. Our state may be surplus in cement but there is no reason why extra cement should be thrown where it is not required.

Recently we have on the market high strength rods with increased bond strength created by ribs (Tis. strong-40, tor steel and GRIP bar). High strength steel requires high strength concretes for maximum efficiency and economy. For prestressed concrete works controlled concrete mix is a necessity. For water retaining structures a poorly designed concrete laid improperly can never be improved with water-proofing agents for plasters.

Again control at site to maintain the specific proportions at all times is necessary. Depending on the possible errors at site we can fix up the control conditions and alter the design strength suitably allowing for a reasonable tolerance. (Table-7).

Depending on the purpose for which the concrete is used, we have varieties of concrete (i) Heavy weight concrete (Nuclear container structures) (ii) Light weight concrete (Partition walls, filler walls, Cladding) (iii) Air entrained Concrete (Dam Construction) (iv) Vaccum concrete (roads, Hume pipes), (v) Ready mixed concrete (Works in crowded localities), (vi) High strength concrete (prestressed concrete works), (vii) Normal Concrete (R.C. Works, filler, levelling course), (viii) Gap graded concrete (cheap, high strength concrete) (ix) Prepakt concrete (repairs to dam), (x) shortcrete (tunnel walls) encasement of girders). Naturally the design procedure have to be different as placing, compaction methods and purpose served are all different. We will restrict our discussion to the basis of design for normal concrete and high strength concrete using ordinary portland cement and natural aggregate.

PURPOSE OF PASTE AND AGGREGATE.

The three basic requirements in concrete mix design are workability, strength and economy. The cement paste serves to fill the voids between the inert particles providing lubrication in fersh state and water tightness and strength in the hardened state. The aggregate provides a cheap filler for cementing material reduces the volume changes in the hardening process and in the working range resists the action of applied loads. In the high strength concrete, the friction between the aggregates also contributes for the strength of concrete. The mix design thus resolves into finding the relative proportion of cement, fine aggregate, coarse aggregate and water. By solid volume 65—75 percent is made up of aggregate and 16 percent by water, and the rest by cement. The cost of cement is 8 to 10 times the cost of aggregate (bulk volume) and hence effort should be made to minimise the use of cement.

VARIABLES IN PROPORTIONING.

Four variables completely define fresh concrete mix and they are so interrelated that if any three is fixed, fourth automatically can have only one value. (1) Water/Cement ratio (2) Aggregate/Cement ratio (3) Overall grading (4) Workability of fresh Mix. (Fig. 1).

WATER/CEMENT RATIO.

Duff Abram's Law states that for any given condition of test, the strength of a workable concrete mix is dependant only on Water/Cement Ratio. (Table 4). Cement requires about 25 percent of its weight for full hydration and does not combine with more than half the quantity of water in the mix. Any extra water simply leaves voids, decreasing the strength. Hence thinner the paste, less the strength. Depending on the condition of environment, a maximum Water/Cement ratio is fixed from durability conditions. (Table-5).

AGGREGATE/CEMENT RATIO.

Highest aggregate content using the biggest size of aggregate possible (Table-1) with minimum fine aggregate to serve as a lubricating film and fill the voids should be arrived at. The strength is not directly affected by the aggregate shape, type, surface texture or the richness of the mix (viz.) Aggregate/Cement Ratio.

Grading.—Grading does not affect the strength directly but for workability, the grading should be such that the total surface area of particles and the volume of voids in the fully compacted state are the least. Hence the grading using the lowest proportion of fine aggregate giving the least surface area will be stronger for the same workablity.

Workability.—The compacting factor index developed by Road Research Laboratory (U. K.) is a measure of the workability. The slump though not related to the workability can still be a rough guidance (Table 8). Depending on placing and compacting methods, the workability can be very low, medium or high.

Classification of Methods of Design—

1. Arbitrary selection-method widely used in departments and private works.

2. Minimum void technique-useful for gap graded concrete.

3. Maximum density method-high conformity required.

4. Sieve analysis technique-Fineness Modulus method, Road Note—4 method, American Concrete Institute Method (conformal methods).

5. Surface area method—Kennedy's method (highly scientific method).

EARLY METHODS :—

In the arbitrary methods, the Coarse Aggregate/Fine Aggregate ratio is specified to lie between $1\frac{1}{2}$ to $2\frac{1}{2}$ by volume assuming maximum density and minimum voids are produced by this combination. Ratios of 1 : 4 : 8, 1 : 5 : 10, 1 : 2 : 4 etc., all by volume are arrived at by this technique and the water content is left to the mixer driver so that the concrete produced is an uncertain building material. In the volume-batching the quantity of the material depends on the size of the measuring box and the unrealiable personal factor. When we have gone for weight measure for rice it appears even more reasonable to go for weigh batching in the manufacture of a structural material like concrete.

The minimum void method aims at finding the void content in the respective aggregates and filling the same with lesser size constituents, expecting a dense concrete by such combination. Water is added to give necessary workability and if the aggregates are not properly graded, this results in low strength concrete mixes.

The maximum density method evolved by Fuller in 1907 is based on the proposition that maximum density can be expected for the concrete if the solid constituents (Aggregate and Cement) are composed of particles varying in size more or less according to a regular parabolic law. A/C ratio is to be fixed by trail mixes and water is added to taste and this conformity does not necessarily give best result for all conditions.

FINENESS MODULUS METHOD.

In 1918 Duff Abram found that aggregate not conforming to the Fuller's curve but having a certain mean value of size of particles are fairly good for concrete making. He used the fineness modulus as the index for selection and grading. The fineness modulus of aggregate which is calculated by adding the percentage by weight retained on a series of sieves and dividing by 100, represents in fact the average size of particles measured not in seieve sizes but in terms of size fractions. Based on experience, certain range of values of fineness modulus for fine aggregate and coarse aggregate as well as mixed aggregate have been found which give good workablity with minimum quantity of cement (Table 3). If the fineness modulus of the mixed aggregate is more, the mix will be harsh, while if it is less the mix will be uneconomical.

Workability depends on the amount of water in the mix and as per Walsh for constant grading and workability, the amount of free water (excluding that absorbed by the aggreggate) required in the mix is constant and is independent of the amount of cement (Table 6). Hence if Water/Cement ratio, if fixed based on Duff Abram's law and the amount of water is fixed by Walsh Law, the balance will be made up by fine and coarse aggregates.

Their proportion is given by

$$P = \frac{(A—B)}{(A—C)} \times 100$$

A19

Where P is ppn. of Fine Aggregate/Combined Aggregate.

A, B and C are the F. Ms. of C.A., combined aggregate and F.A. respectively.

After satisfying that the locally available F.A. and C.A. are having F.M. lying in the ranges stipulated, they can be combined to have F.M. in the necessary range. For lean mix, aggregate grading increases workability and in rich mixes, the effect of grading is unimportant. Least surface area is obtained by going for max. size of aggregate. Since water required for workability is less, this results in most economical mix.

Particles are three dimensioal and sieve openings are two dimensional in character. Hence the method of separating by sieving F.M. index, etc., are at best two dimensional solutions for a three dimensional problem. Hence these methods can be used as guides and workability checked by infinite number of gradings and hence some experience is necessary in choosing and combining the aggregates.

TYPE GRADING CURVE METHOD :

This method advocated by Road Note-4 of R.R.L. (U.K.) and adopted extensively in British practice, follows W/C for strength. For a fixed grading, W/C ratio and required workability (Table-2) there can be only one value of A/C ratio. As the specific surface of the aggregate on shape, texture and grading, based on experiments, values of C/A are tabulated separately for rounded, regular, angular and crushed aggregate for various gradings and workability (Table-11).

This method requires the course and fine aggregate to be so combined that the mixed aggregate has the same percentage retained on various standard sieves as have been found to give the best results. Four typical garding curves for aggregate are given depending on the maximum size aggregate (Figure—2). Curves (1) and (4) represent the limits of formal continuous grading, with curve (1) specifying 30 percent material 3/16" down and curve (4) specifying 48 percent material 3/16" down. The specific surface of material conforming to curve (1) will be less than that of curve (4) and hence A/C will be comparatively more for above (1) for the same workability. If W/C is less (Rich mix) adoption of grading (4) lands in dry segregation and if W/C is more (lean mix) adoption of grading (1) lands in wet segregation. Hence depending on water content used and the relative surface area of fine particles in the grading, the suitable range must be selected. The various sizes of aggregate must be combined to conform as nearly as possible, the point corresponding to sieve of 3/16" atleast made equal. When the mix is designed, it must be tested for workability by trial mixes.

Surface Area Method (Kennedy's Method) :—

For the same film thickness workability increases with the consistency of the paste. Depending on the workability and the W/C fixed from strength consideration, thickness of film coat is fixed. The paste is required for two purposes : (*a*) for enveloping the aggregate with a thin film, (*b*) to fill up the voids in the combined aggregate. It can be proved that the cement required of concrete :

$$Y = \frac{1}{(a + x) + \frac{NW}{W_s}} \quad \text{Where } N = \frac{a + x}{\frac{KS}{10^4} + \frac{W_s - W}{W_s}}$$

Where x = Volume of Water required for 1 c.ft. of cement.

a = solid volume of cement.

K = thickness of film in 10^{-4} ft. unit.

W = weight/c. ft. of aggregate (bulk volume)

W_s = Weight/cft. of aggregate (solid volume)

S = Specific surface in S. ft./1 c. ft. of aggregate (bulk volume.)

$$S = \frac{6W}{W_s} \Sigma \frac{P}{d}$$

Where P is the percentage by weight of material retained between two consecutive sieves where 'd' is the diameter of aperture on lower sieve.

For an economical mix Y should be the least, that is N should be large (i. e.) W should be large and S should be small. For various proportion by weight of FA/CA, determine W,S and hence Y and use the proportion giving the least Y. This proportion combines strength, workability and economy.

Other Methods :—

Richard Talbot method is based on the proposition that strength of concrete depends upon strength of mortar and aggregate content is fixed based upon shape, grading and compaction. The ACI method fixes the solid volume of Coarse Aggregate based on volume of dry rodded aggregate per unit volume of concrete based on F.M. of F.A. and maximum size of aggregate (Table 9). In the trial mix method, for a fixed W/C ratio and workability various amounts and combinations of FA/CA are tried and the optimum combination requiring the least cement is selected by trials.

Design of high strength concrete :

In the design of high strength mixes, as the strength depends not only on the consistency of the paste (i.e.) Water/Cement ratio but also on the total aggregate content (i.e.) Aggregate/Cement ratio, due to sharing of the load by friction between particles, the design charts are drawn by interaction diagram working between strength and workability. Dry mixes with low workability can be produced by a vide variation of A/C ratios (2.5 to 5.5) but having the same strength. In the high workability regions, richer mixes are stronger. It is better to have leaner and dry mixes and take more trouble to compact them if cement is to be reduced.

The following points are worth noting in the designing and placing of concrete to attain the requisite properties.

1. *Coarse Aggregate (Size, Shape and grading).*

Use locally available aggregate, the biggest size in keeping with the size of the member and cover requirements. Rounded aggregates have less surface area and hence result in saving in cement for the same strength of Concrete (Water require less and hence for fixed W/C, cement less).

Don't break the rounded aggregeate to make it angular and cube like. Don't break the crushed aggregate to still finer size expecting higher strength. The surface area becomes more and hence for maintaining the same workability higher water content is required. Prefer a continuous grading rather than asking for one particular size.

2. *High Strength Concrete* :

For high strength concrete as the strength depends on strength of matrix and also on friction between aggregate and matrix, crushed aggregate and high A/C ratio preferred. ¾" or ⅜" maximum size aggregate give the same strength for same workability.

Even for normal concrete where high flexural strength is required, angular or irregular aggregate must bre preferred.

3. *Specification for Water.*—

Avoid alagae infested water for making Concrete as 50 percent loss in strength due to air entrainment can be expected. Brackish and sea water can be used for plain concrete if strength does not fall by more than 10 percent and allowance made in design (if efflloracence can be tolerated). We can be touchy

about cleaning water but can be liberal about curing water. For reinforced and prestressed concrete works don't use sea water.

4. *Workability.*—

Increase the workability by increasing the richness of the mix than having a dry lean mix and trying to compact it with grater vigour, if sand is more, more cement is needed for consistency and if sand is less, the mix will be harsh. Sand content of 35 percent appears to be optimum for normal workable concrete.

Use slump test for faulty batching and compaction factor test for workability.

Do not allow the worker to increase the water to make the concrete flow. For full hydration 0·26 percent water by wt., is required and as only 50 percent of free water usually combines, W/C >0·50, leave air voids, decreasing the strength.

5. *Placing and Segregation.*—

Concrete is perishable like vegetable but can be kept from setting by agitating upto 6 hours. Avoid dry segregation during compatcion by going for coarser grading for rich dry mixes (using less sand). Avoid wet segregation during handling by going for finer grading with higher sand content for lean wet mixes.

6. *Mixing, Compaction and Curing.*—

Mixing for 20 revolutions extending for 1— 1½ minutes is adequate, giving about 24 batches/hour. Don't use poker vibrator for slabs less than 6" thick, Poking at 2' intervals for heights not grater than 2 ft. extending for 1/2 to 2 minutes for stiff mixes and 20—40 sec. for workable mixes, is a good practice. Curing consists of maintaining water content at specified temperature. Hence specify the time of removal of forms and props based on strength gained. Dead load and possible live load should be carried with factor of safety of 2 (for beams strength not less than 2,000 psi for props removal).

7. *Concrete for Water Retaining Structures.*—

Use least water required from workablility consideration. Aggregate/Cement not greater than 6. The Coarse Aggregate must be made up of particles not of one size but continuosuly graded. Use the minimum sand necessary. Compact well and cure for not less than seven days thoroughly (66—75 percent strength). Thickness of wall or slab not less than 5".

8. *Specification and Testing* :

While sending cubes for testing, do not over dry specimens. They are apt to give high strengths. For standard tests, saturated wet condition which give conservative values are preferred.

Do not over specify. To prevent lean mixes for R.C. A/C not greater than 8 and for shrinkage limits A/C not less than 3. (7,500 Psi) and not less than 4·5 (5,000 psi) are advisable limits.

Conclusion.—

The concrete used must be designed for the job weigh batching must be insisted for concrete making. The local Engineer is the best judge to design the concrete based on the local aggregate control conditions and vibration equipments and workmanship available at the site.

APPENDIX.

Table 1.—Maximum Size of aggregate recommended by ACI 613—54.

Min. Dimension of section in inches.	*Max. size of aggregate in inches.* *B.C. walls.*	*Reinforced walls.*	*Heavily reinforced slabs.*	*Lightly reinforced or unreinforced slabs.*
(1)	(2)	(3)	(4)	(5)
2½ —5	½—¾	¾	¾—1	¾—1½
6—11	¾—1½	1½	1½	1½—3
12—29	1½—3	3	1½—3	3
30 or more	1½—3	6	1½—3	6

Table 2.—Uses of Concrete of different degrees of workability RN—4.

Degree of workability.	*Slump in inches.*	*Compaction factor.* *Small.*	*Big.*	*Remarks.*
(1)	(2)	(3)		(4)
Very low	0—1	·78	·80	Vibrated concrete—Roads.
Low	1—2	·85	·87	Mass concrete with simple reinforcement—Roads.
Medium	2—4	·92	·935	Normal reinforcement work—Building.
High	4—7	·95	·96	Not suitable for vibration.

Table 3.—Recommended fineness modulus limits for F.A. C.A. mixed aggregates.

Max. size of aggregate.	*F.A.*	*¾"*	*1½"*	*3"*	*6"*	*Remarks.*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
F.M.—Min.	2	6	6·9	7·5	8·0	....
(F.A. & C.A.)—Max.	3·5	6·9	7·5	8·0	8·5	....
F.M. (Combined Agg.)—Min.	..	4·7	5·4	5·8	6·5	Higher W/C.
A/C—5 (Combined Agg.—Max.)	..	5·1	5·9	6·3	7·6	Lower W/C.
A/C—9—Min.	..	4·3	5·0	..	..	Higher W/C.
Max.	..	4·7	5·5	..	..	Lower W/C.

***Table 4.—W/C Ratio—Strength relationship at 28 days—RN—4.**

W/C by Wt...	0·35	0·40	0·45	0·50	0·55	0·60	0·65	0·70	0·75	0·80
Strength (psi)	7,500	6,700	6,000	5,300	4,500	4,000	3,500	3,100	2,800	2,500

Strength at 3 months and 1 year are 25 per cent and 67 per cent greater than 28 days strength.

Strength at 7 days and 14 days are 33 per cent and 25 per cent less than 28 days strength.

* The table of Strength refers to Cements Conforming to B.S.S.

Table 5.—Max. W/C for durability for conditions of exposure ACI 613—54.

Conditions of exposure.	*W/C ratio.* *This section less than 12"*	*Mass concrete.*
(1)	(2)	(3)
Regular wetting and drying ..	0·45	0·55
Continuously under water,	0·55	0.65
Normal outside exposure Buildings bridges not coming in groups above.		
(a) Severe climate	0·50	0·60
(b) Temperature climate	0·65	0·65
(Interior—Strength basis only)		

Table 6.—Water required for 100 c.ft. of concrete 3" slump.

Maximum size of aggregate (in inches).	¾	1½	3	6	*Remarks.*
(1)	(2)	(3)	(4)	(5)	(6)
Rounded aggregate	1,149	1,037	926	828	5 per cent by wt. of agg. + 30 per cent by wt. of cement.
Irregular, angular, gravel or crushed aggregate.	1,241	1,122	1,019	894	For vibrated concrete, reduce water by 20 per cent. Decrease water by 5 per cent to decrease slump to 1".

Table 7—Min. and average Crushing Strength for Different Conditions.

Condition of control.	*Min. Adv. strength.*	*Standard deviation.*	*Co-efficient of variation.*
(1)	(2)	(3)	(4)
	PER CENT.		
Excellent-weigh batching graded, aggregate in 3 Sizes, moisture, determination, constant supervision, controlled curing.	90	400	5·0
Good—weigh batching , graded aggregate moisture determination, constant supervision.	75	500	10·8
Fair—weigh batching, graded aggregate W/C not controlled, occasional supervision ..	60	750	17·2
Poor—Volume batching—Little Supervision	40	1,000	25·[illegible]

Table 8.—Recommended Slumps.

Description.	*Max.* (IN INCHES)	*Min.*
R.C. Foundation walls and footings	5	2
Plain footings, substructure walls	4	1
Slabs, beams and R.C. walls	6	3
Building columns	6	3
Pavements	3	2
Heavy construction	3	1

TABLE 9.—VOLUME OF C.A/UNIT VOLUME OF CONCRETE.

Maximum size of aggregate in inches.	*Volume of dry rodded agg./Unit Vol. of concrete for different F.M. of F.A.* 2·40	2·60	2·80	3·0
(1)	(2)	(3)	(4)	(5)
⅜	0·46	0·44	0·42	0·40
¾	0·65	0·63	0·61	0·59
1½	0·76	0·74	0·72	0·70
3	0·84	0·82	0·80	0·78
6	0·90	0·88	0·86	0·84

TABLE 10.—STRENGTH REQUIREMENTS.

Description.	*Strength as 28 days (psi).*
(1)	(2)
Prestressed concrete	above 6,000
High strength reinforced concrete	3,500—5,000
Tanks, reservoirs, precast thin sections	3,500—5,000
General R. C. buildings, bridges, silos, Pavings, foundations for heavy loads.	2,500—3,500
Mass concrete—dams, retaining walls	2,000—2,500
Mass concrete roughest type	1,500—2,000

TABLE 11.—A/C REQUIRED FOR DEGREES OF WORKABILITY WITH DIFFERENT GRADING OF ¾" CRUSHED AGGREGATE RM—4.

Degree of workability.	*Very Low.*				*Low.*				*Medium.*				*High.*			
Grading of aggregate (Curve No. on Fig. 2.)	1.	2.	3.	4.	1.	2.	3.	4.	1.	2.	3.	4.	1.	2.	3.	4.
0·35	3·2	3·0	2·9	2·7	2·7	2·7	2·5	2·4	2·4	2·4	2·3	2·2	2·2	2·3	2·1	2·1
0·40	4·5	4·2	3·7	4·5	3·5	3·5	3·2	3·0	3·1	3·2	2·0	2·9	2·9	2·9	2·8	2·6
0·45	5·5	5·0	4·6	4·6	4·3	4·3	4·2	3·9	3·7	3·7	3·4	3·3	3·5	3·5	3·2	3·1
0·50	6·5	5·8	5·4	5·0	5·0	4·9	4·5	4·3	4·2	4·2	3·9	3·8		3·9	3·8	3·5
0·55	7·2	6·6	6·0	5·6	5·7	5·4	5·0	4·8	4·7	4·7	4·5	4·3			4·3	4 0
0·60	7·8	7·2	6·6	6·3	6·3	6·0	5·6	5·3		5·2	4·9	4·8			4·7	4·4
0·65	8·3	7·8	7·2	6·9	6·9	6·5	6·1	5·8		5·7	5·4	5·2			5·1	4·9
0·70	8·7	8·3	7·7	7·5	7·4	7·0	6·5	6·3		6·2	5·8	5·7			5·5	5·2
0·75			8·2	8·0	7·9	7·5	7·0	6·8			6·2	6·1			5·8	5·7
0·80					..	..	7·4	7·2			6·6	6·5			6.1	6·0
0·85					..	..	7·8	7·2	..		7·1	6·9			6·4	6·3
0·90					..	..	..	..			7·5	7·3			..	6·7
1·95					..	..	..	..			8·0	7·6			..	7·0
1·00					..	..	..	..	..						..	7·7

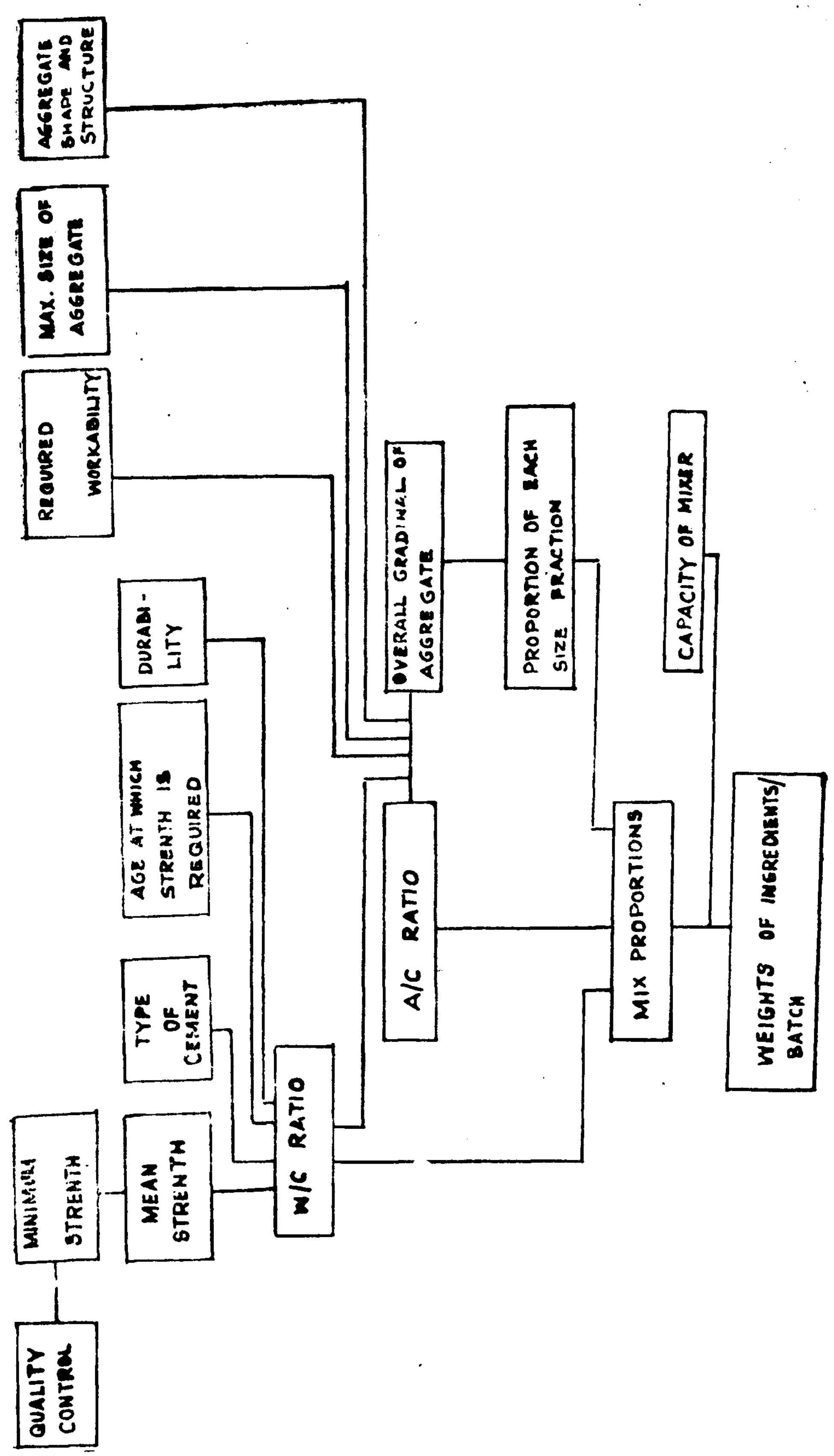


FIG. 1 FLOW DIAGRAM FOR MIX DESIGN.

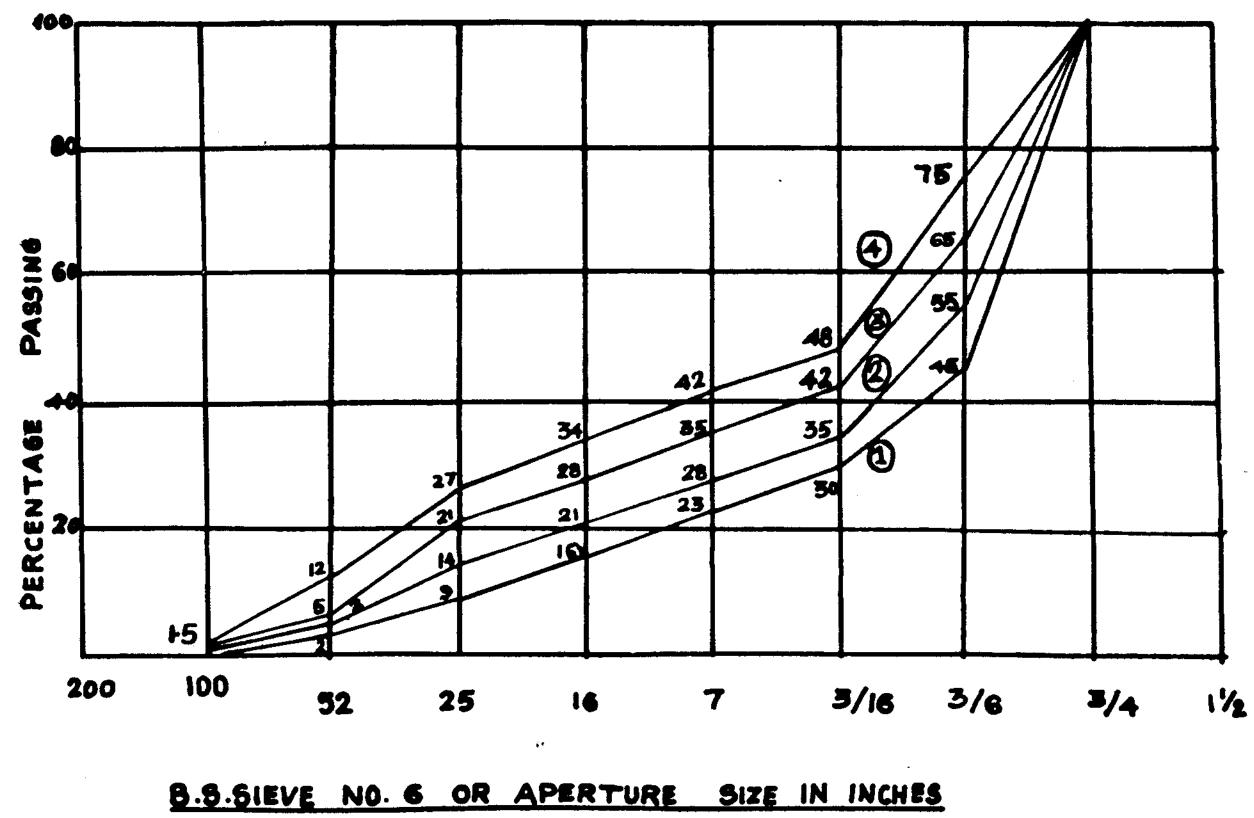


FIG. 2. TYPE GRADING CURVES FOR ¾" MAXIMUM SIZE AGGREGATE

SAND CEMENT DRAIN CHUTES FOR GRAND ANICUT CANAL

BY

THIRU K. RAMALINGAM, B.E., A.M.I.E. ‡

1. *Introduction.*

Chutes are essential for preserving the banks of canals and tanks from frequent erosion by rain-water. Where these banks also carry a road-way the chutes become indispensible. Chutes are generally built with the locally available material to effect economy. This paper discribes how, strong and cheap chutes could be made out of locally available sand and cement. Thanjavur district has no stone-quarry and her stock of bricks is too limited to be adequate even for building residential and irrigation structures. Therefore, in this district sand-cement is turning out to be a versatile and cheap material for use in revetments and aprons laid to protect the water courses and cross-masonry structures.

2. *Brief Description of G.A. Canal.*

This canal was excavated in 1930 from Grand Anicut across Cauvery, where from the other two rivers Cauvery and Vennar also branch off. This canal ranks as the first and the largest of the ventures in modern canal construction undertaken in South India, having been designed by Col. Ellis. During that period, it was not a recognised practice to line the canals. The canal had shown, over these 40 years, evidence of its weakness against the ravages of rain being susceptible to erosion. To prevent this erosion, the sides had been reveted wherever necessary year after year. The main at either sides of the banks. Gullies invade from both sides causing wide deep cleavages dividing the banks almost into two. This necessitates large expenditure for maintenance. To prevent formation of such gullies chutes have been built with sand-cement mortar.

3. *Description of Banks.*

The ayacut under this canal, as in Cauvery and Vennar rivers is covered by Intensive Agriculture Development Projects. Frequent breaches used to take place in the canal banks upsetting the programmes under these projects. The Government of Tamil Nadu sanctioned about Rs. 54 lakhs for strengthening the canal banks. Under this special grant the banks were widened to the following standards in the first 40 kilometres of the canal :—

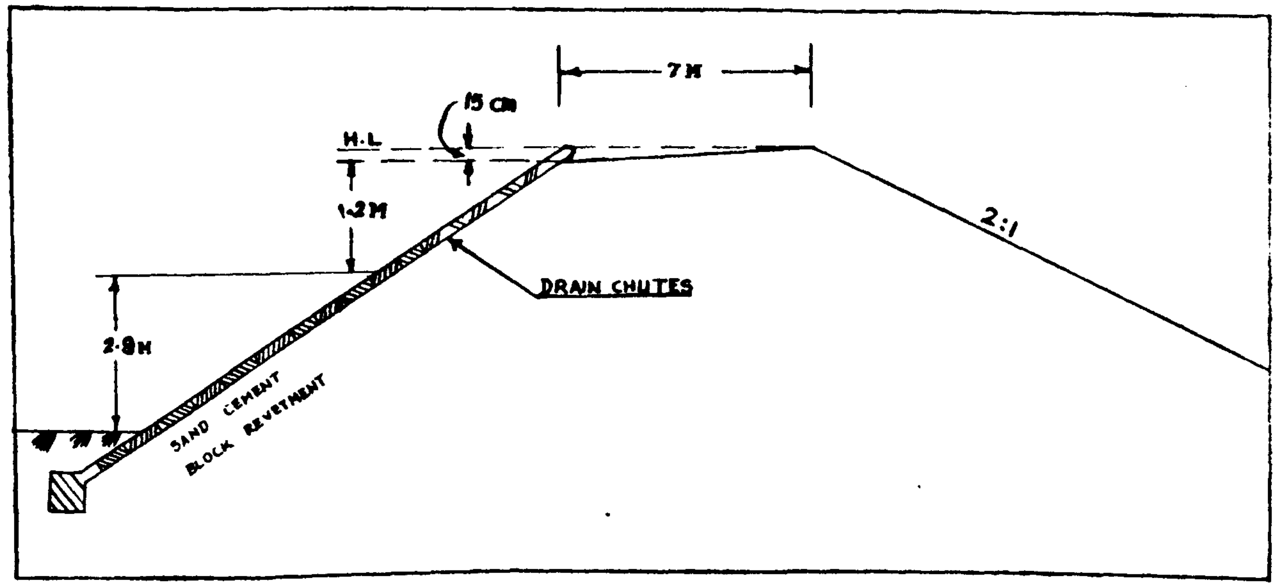
Top width of left bank—7·0 metres.

Top width of right bank—4.5 metres.

Front slope—$1\frac{1}{2}$ to 1.

Rear Slope—2 to 1.

The top of bank was not kept flat with a camber at centres as is usual with all banks and roads. The front edge of the canal is kept lower than rear edge by about 15 cm.



canal is nearly 160 km. in length and the revetment had been laid over only 30 per cent of its length. Erosion is still continuing and during every year's closure-season, the revetments are being laid in more reaches. The canal is carrying a road-way. This is mostly used by Inspection vehicles and at a few-places public buses also ply. During every monsoon, the rains leave behind a heavy toll of erosion on the banks. The soil being only sand-loam deep gullies are formed

This slope is given to induce rain water to flow into the canal rather than outside. If gullies are allowed to form on the rear side of the canal, they will be long and deep. If gullies form on the front side, they will be short and shallow. Drain chutes were placed along this short sloping surface. After strengthening work, canal banks were allowed to settle and toweather one monsoon and suffer gullies. Chutes were placed wherever gullies actually accured.

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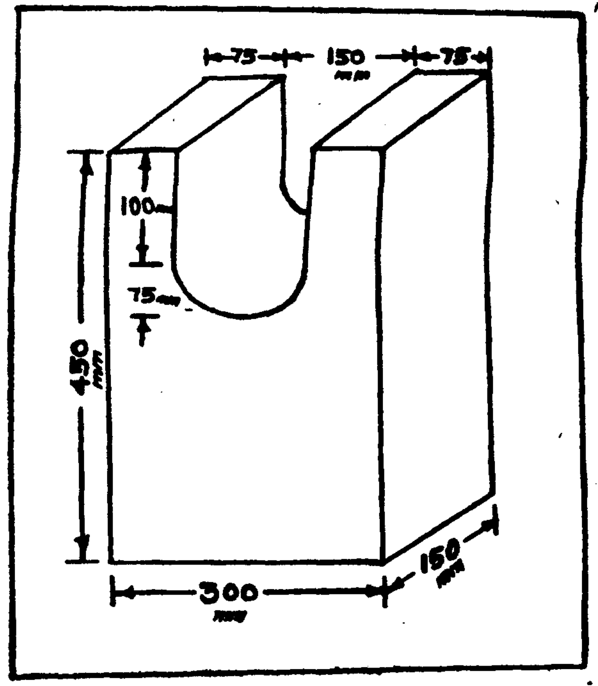
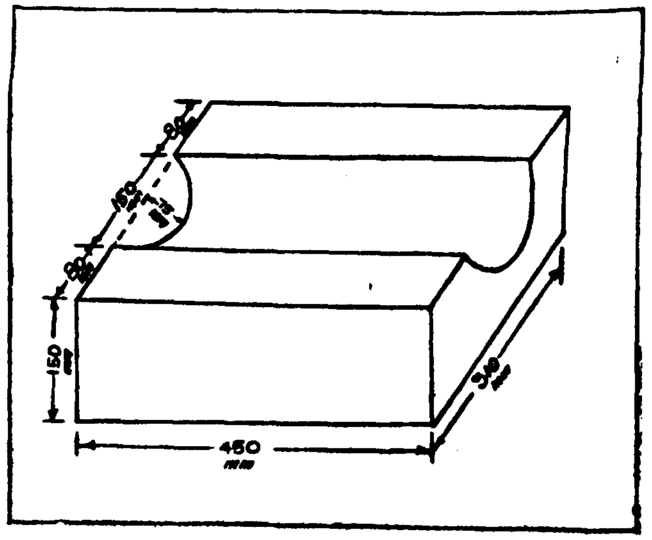
4. *Description of chutes.*

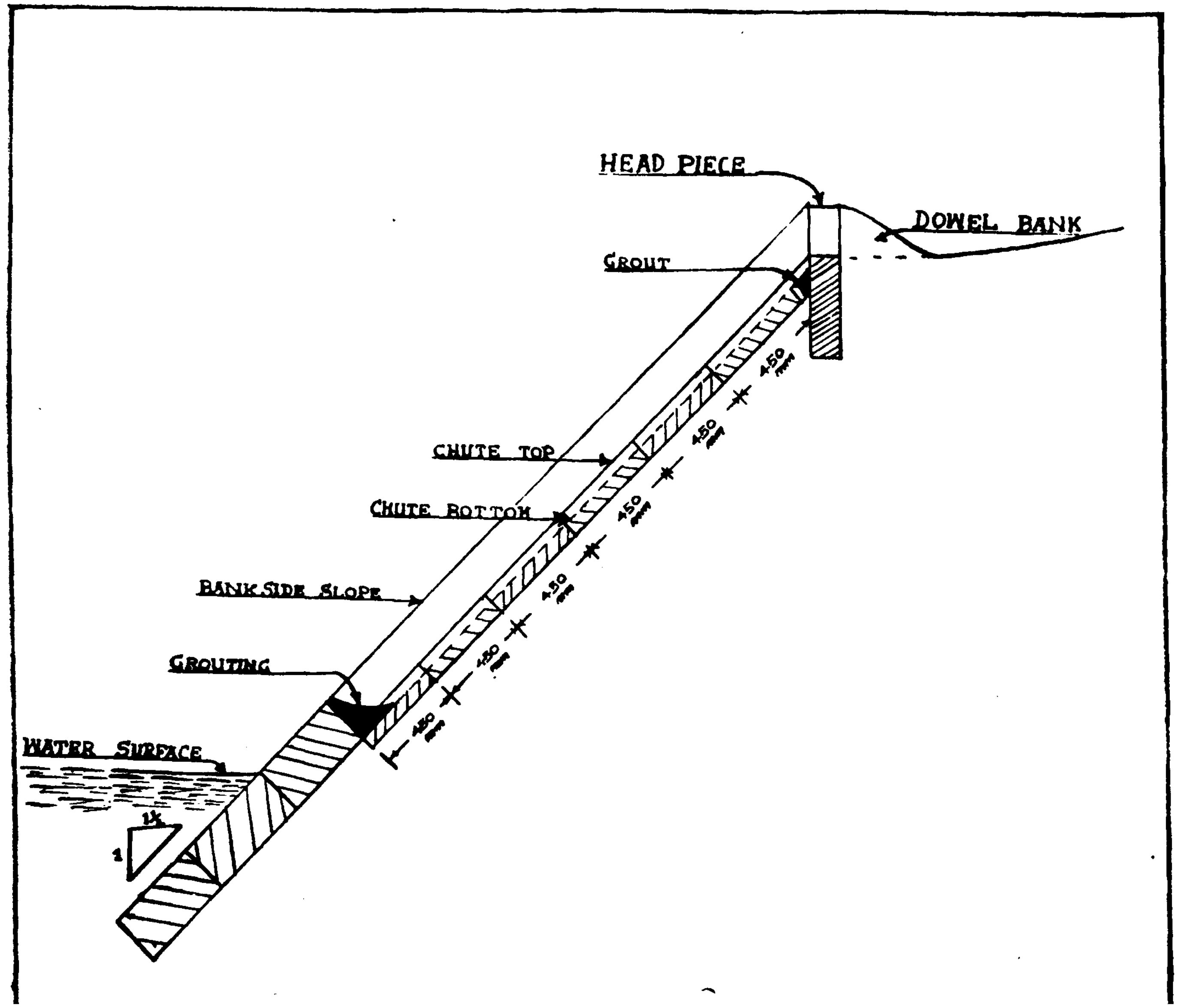
Chutes were cast from sand cement mortar of ratio 1:6. They consisted of 2 elements (*a*) head-piece and (*b*) drain-piece. Head-piece was of size 450 × 300 × 150, with a U shaped cavity at one short edge. Drain-piece was of size 450 × 310 × 150 with a semi-circular groove of 150 mm. diameter in the centre. These were arranged in the following pattern.



Head-piece



Drain-piece



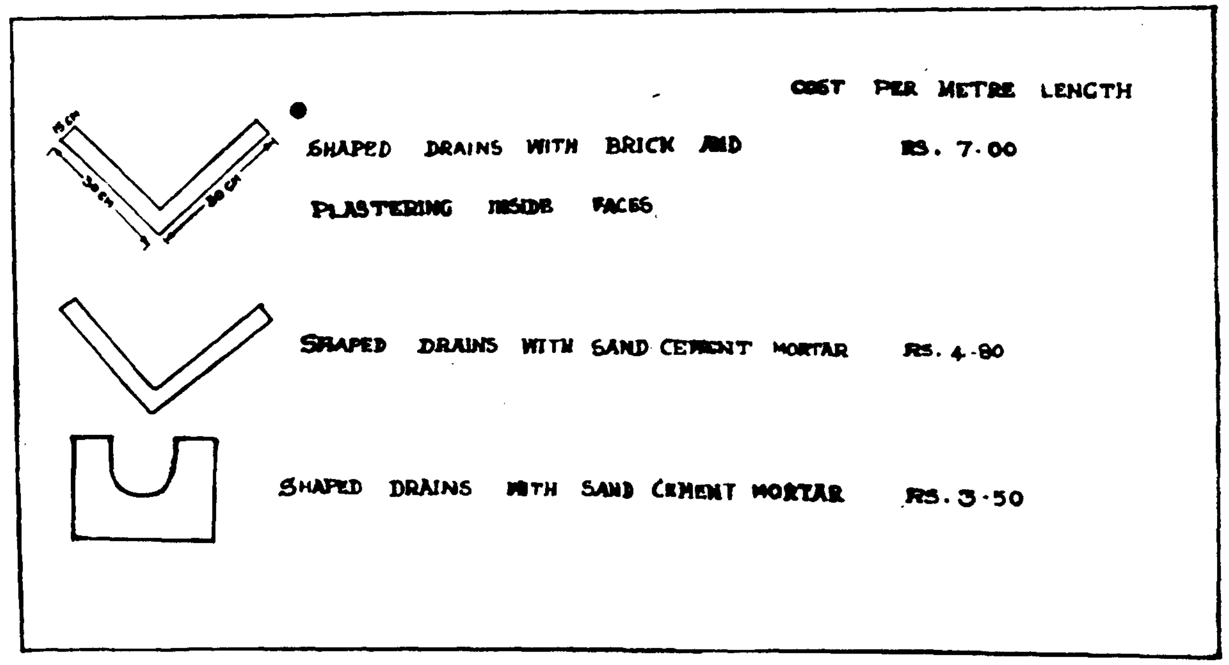
The head piece was kept in the line of the dowel bank along the front edge of the bank. The sill of the U in the head piece was kept about 2 cm. below the bank level in front of the chute. Chutes were arranged upwards from the top of the revetment. The gross length of chutes ranged from 2 metres to 10 metres depending upon the levels of the banks. The clearance between the last chute and the top edge of revetment was grouted and finished with mortar in a curve shape for easy flow. The clearance between the head-piece and the first chute was also grouted. The joints between the chutes were not grouted.

5. *Performance.*

The chutes were watched during one monsoon. They behaved very well. Drain-pieces at a few locations settled due to poor compaction of soil under them. They were reset after consolidation. After erection of these chutes, the spacing between adjacent chutes were found to range from 50m. to 80m depending on the lie of the banks. Generally drain chutes are built in V or U shapes with 30 cm. depth and 60 cm. width using either random rubble or brick. Compared to this general practice, the dimension of the U drain adopted with sand-cement chutes might look rather small. This fear was disproved by actual experience. During heavy rains the chutes were actually observed to carry their discharges with safe free-boards of over 20 mm.

6. *Relative Merits.*

Chutes with different materials and shapes were tried and the following were approximate rates per metre length for the finished work including earth-work required for laying them.



These chutes are easy to cast and handle. There is no need to convey any material except cement. These are easy to repair and to shift. No skilled labour is necessary as any semi-skilled labourer can cast or arrange the piece after a few trials. On large works part of cement can be displaced either by lime or by fly-ash. Through proper preplanning and precasting, these chutes lend themselves to very quick and cheap work.

FIELD LOAD TEST ON A FULL SCALE MODEL OF HYPERBOLIC PARABOLOID FOOTING.

BY

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1.0. *Synopsis* :—

The paper presents an actual full scale test on a Hypar footing adopted in the Construction of Government Arts College at Ramanathapuram. The footing was of size 180 c.m. ×160 c.m. with a rise of 50 c.m. It was 15 c.m. thick with reinforcement 10m.m. ϕ bars at 20 c.m. c/c both ways. The test was carried out upto a load of 45 tons which caused a settlement of 3.95 m.m. only.

2.0 *Scope of the Test* :—

The test was carried out in connection with the proposal to adopt hypar footings in the construction of Government Arts College at Ramanathapuram. The safe bearing capacity of the soil at site was 0.7 t/sft with a factor of safety of two. The water table was at a depth of 8 ft. from ground level during the test. Since it was expected that the water table may still rise during winter, it was proposed to carry out the test in two cycles allowing the footing to stand against any seasonal changes. It was proposed to carry out the test upto twice the design load i.e. 40 Tons.

3.1. *Details of the test* :—

The hypar footing laid for being tested has the following dimensions and specifications.

Size 180×160 c.m.

Rise 50 c.m.

Edge Beam 15× 30 c.m.

Shell Thickness 15 c.m.

Reinforcement :—

Shell—10 m.m. ϕ at 20 c.m. c/c both ways.

Edge beam—12 m.m. ϕ— 4 Nos.

Inclined ribs—10 m.m. ϕ—4 Nos.

Stirrups—6 m.m. ϕ at 30 c.m. c-c. for beams

A, R. C. Pedastal 40 c.m. × 40 c.m. has been provided for the 20 c.m.×40 c.m. column and the reinforcement anchored for a distance of 30 D.

3.2. The earth was shaped from the undisturbed soil to form the hyperbolic paraboloid surface and the edge beams rest at a depth of 8 ft. from the G.L. The finished surface was covered with a layer of 1 : 5 : 10 plain concrete, 5 c.m. thick. The reinforcement bars were placed such as to lie in the middle of the shell thickness. Concrete was then poured to the required thickness of 15 c.m. and finished correct to the shape by moving the straight edge.

Date of casting of footing—12th August 1970.

First Cycle of load test on—21st September 1970.

Second Cycle of load test on—23rd January 1971.

4.1. *Test Procedure* :—

During the first cycle of test the load was applied through a 50 tons hydraulic Jack. The necessary reaction was provided by a platform loaded with steel rods and sand bags, to about 40 tons. Deflectometeres were fixed to measure the settlements. The load was applied in increments of 5 tons and the Deflectometer readings were recorded at every increment of load. The design load of 20 tons was maintained for a period of 12 hours and the deflectometer readings were recorded at every one hour. At this stage the test was discontinued and the footing was allowed to stand against any seasonal changes taking place in the soil.

4.2. *The Second Cycle of load test* :—

The second cycle of load test was resumed after four months. Now the load on the reaction platform was about 55 tons. A load of 45 tons was applied in stages of 5 tons and the deflections noted. The test could not be continued further since at this stage cracks were formed adjascent to the pedastal of the column and deflectometer thrown, out of order. Even though the ultimate failure of the soil has not taken place, the applied load of 45 tons exceeded the design ultimate capacity of 40 tons.

5.0 *Test Results* :—

The load settlement curve for the two tests combined together is as in the figure. During the first cycle of test the settlement was 1.31 m.m. against the calculated value of 2.28 m.m. In the second cycle of test under the load of 45 tons the settlement was 3.95 m.m., the calculated value being 4.63 m.m.

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6.0. Concluding Remarks :—

At the load of 45 tons on the footing, the bearing pressure on the soil is 1.45 tons/sq.ft. Although from plate load test, the ultimate bearing capacity of the soil has been predicted as 1.4 t'sft., the failure of the soil under the footing could not be predicted because of the steady trend of the Load-settlement curve. On the basis of the settlement criterian for an angular allowable distortion of 1/500 of frame the ultimate load that the footing can stand worked out to 56 tons which corresponds to a settlement of 6.8. m.m. Here for the applied load of 45 tons the settlement was only 3.95 m.m. This proves the efficiency of the hypar footing which is less susceptible for settlement due to the confinement of the soil below the shell portion.

DESIGN OF HYPERBOLIC PARABOLOID FOOTING.

Load—20t. SBC of soil 7 t/m². shell thickness 15 cm.

Area of base required $= \frac{20}{7} = 2{\cdot}86\text{m}^2$

Adopt a footing size of 1.8 m. × 1.6 m. then x = 0.9 m, y = 0.8 m.

Adopt a rise of 0.5 m.

$$\text{then } k = \frac{0{\cdot}5}{0{\cdot}8\text{x}0{\cdot}9} = \frac{0{\cdot}5}{0{\cdot}72} = \frac{1}{1{\cdot}44}$$

$$\therefore \text{ shear/m. run, } t = -q\frac{1}{2k} = \frac{7{,}000 \times 1{\cdot}44}{2} = 5{,}040 \text{ kg.}$$

In determining the shear force t, the value is obtained by taking $q = 7\text{t/m}^2$ as the verctical upward reaction of the soil. the maximum values of normal forces "n_x" and "n_y" are found by assuming $q = 7\text{t/m}$ acting normally upward on the shell surface.

$$\text{Shear stress in concrete.} = \frac{5040}{15\text{x}100} = 3.36 \text{ kg/cm}^2$$

10 mm. rods at 20 cm. c/c shall be provided in both directions and this will completely take up the shear stress.

Direct Stresses :—

$$\text{Max. } n_x = \frac{3q}{2} \text{ y. Sinh}^{-1} \frac{x}{u}$$

$$n_y = \frac{3q}{2} \text{ x. Sinh}^{-1} \frac{y}{v}$$

$$u = \sqrt{1/K + y^2} \qquad v = \sqrt{1/K_1 + x^2}$$

$$= \sqrt{1{\cdot}44^2 + 0{\cdot}8^2} \qquad = \sqrt{1{\cdot}44^2 + 0{\cdot}9^2}$$

$$= \sqrt{2{\cdot}07 + 0{\cdot}64} \qquad = \sqrt{2{\cdot}67 + 0{\cdot}81}$$

$$= \sqrt{2{\cdot}71} = 1{\cdot}648 \qquad = \sqrt{2{\cdot}88} = 1{\cdot}695$$

$$\frac{x}{u} = \frac{0.9}{1.648} = 0.546 \qquad \frac{y}{v} = \frac{0.8}{1.695} = 0.472$$

$$\text{Sinh}^{-1} \frac{x}{u} = \sinh^{-1} 0.546 = 0.5216$$

$$\text{Sinh}^{-1} \frac{y}{v} = \sinh^{-1} 0.472 = 0{\cdot}4554$$

$$n_x = \frac{3 \times 7000}{2} \times 0{\cdot}8 \times 0{\cdot}5216 = 4830 \text{ kg/m.run.}$$

$$n_y = \frac{3 \times 7000}{2} \times 0{\cdot}9 \times 0{\cdot}4554 = 4330 \text{ kg/m. run.}$$

10 mm rods provided in both directions will completely take up normal stresses n_x and n_y.

Design of tie Beam :—

Max. tension in the external tie beam
$= 0.9 \times 5040 = 4536$ kg.

$$\text{Reinforcement required} = \frac{4{,}536}{1{,}400} = 3.24\text{cm}^2$$

In the other direction max. tension
$= 0.8 \times 5040 = 4{,}032$ kg.

$$\text{Reinforcement required} = \frac{4{,}032}{1{,}400} = 2.88 \text{ cm}^2.$$

Provide 4 rods of 12 mm for both tie beams
Area = 4.52 sq.cm.

Beam Section :—

Max. tension = 4,536 kg.

Let AC be the area of concrete section required Assume a composite stress of 14.06 kg/cm².

$$\text{then, } \frac{4{,}536}{\text{AC} + 18 \times 4.52} = 14.06$$

$$\frac{4{,}536}{14.06} = \text{AC} + 18 \times 4.52$$

$$\text{AC} = \frac{4{,}536}{14.06} - 81.36$$

$$= 322 - 81.36 = 240.64 \text{ cm}^2$$

Adopt a section of 15 × 30 cms.
Total load = 20t.
Area of base required = 1.8 m × 1.6 m.

$$\text{Intensity of loading} = \frac{20 \times 1{,}000}{1.8 \times 1.6} = 6{,}960 \text{ kg/m}^2$$

$$\text{Shear force} = \frac{6{,}960 \times 1.44}{2} = 5{,}011 \text{ kg/m. run.}$$

STRESSES IN EDGE BEAM.

Edge beam.	*Horizontal length in m.*	*Vertical rise in m.*	*Inclined length in m.*	*Shear/Unit.*	*Longitudinal force in kg.*	*Horizontal comp. in kg.*	*Vertical comp. in kg.*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
FE/ED ..	0·9	0·5	1·03	2×5,011	10,323	9,019·8	5,011
BE/EH ..	0·8	0·5	0·95	2×5,011	9,521	8,017·6	5,011
AB/GH / BC/HI	0·9	..	0·90	5,011	4,509·9	4,509·9	..
AF/FG / CD/DI	0·8	..	0·80	5,011	4,008·8	4,008·8	..

STATIC CHECKS.

1. Sum of vertical components, 2 (5,011+5,011) = 20,044 kg., which is nearly equal to in coming load 20t.

Hence safe.

2. Sum of horizontal components.

Section BEH,

Tension in AB+GH = Horizontal component of FE.
i.e., 4,509·9+4,509·9 = 9,019·8
9019·8 = 9019·8 Hence safe.

SECTION, FED.

Tension in AF and CD = Horizontal component of BE.
i.e., 4,008·8+4,008·8 = 8,017·6. Hence safe.

3. Moment at column face : *Moment about BH.*

Moment due to external load.

$$= 2 \times 0{\cdot}8 \times 0{\cdot}9 \times 6969 \times \frac{0{\cdot}9}{2} = 4510{\cdot}08 \text{ kg/cm.}$$

Moment due to internal forces.
$= 9{,}019{\cdot}8 \times 0{\cdot}5 = 4{,}509{\cdot}9$ kg.m. Hence safe.

Due to external load *Moment about FD.*

$$\text{Mt.} = 2 \times 0{\cdot}9 \times 0{\cdot}8 \times 6{,}960 \times \frac{0{\cdot}8}{2} = 4{,}008{\cdot}96 \text{ kg.m.}$$

Mt. due to interal forces = 8,017·6 × 0·5 = 4,008·8 kg.m. Hence safe.

REINFORCEMENT IN INCLINED RIBES.

Approximate area of rib at column face
$= 15 \times 30 + 1/2 \times 7{\cdot}5 \times 30$
$= 562{\cdot}5 \text{ cm}^2$.

FE/ED, total compression = 10,323 kg.

$$\text{Compressive stress in concrete} = \frac{10{,}323}{5{,}625} < 20 \text{ kg/cm.}^2$$

BE/EH Total compression = 9,521 kg.

$$\text{Compressive stress} = \frac{9{,}521}{562.5} < 20 \text{ kg/cm}^2$$

In both directions at column face the compressive stress in concrete is less than the allowable value.

Hence, no special provision of reinforcement is necessary. However, 4 rods of 10 mm. can be provided in the rib section.

6 mm. stirrups at 30 cm c/c shall be provided in tie beam reinforcement.

Typical corner reinforcement 12 mm dia. shall be provided for a length of 0·45 m. from corner on either directions.

40 cm. square R.C. pedastal shall be provided. Column reinforcement shall be anchored into the footing for a distance " 30d."

CALCULATION OF ULTIMATE CARRYING CAPACITY OF THE HYPAR FOOTING.

As the footing rests on the sandy clay layer, the calculations are made with respect to the soil at 8 feet depth only. For this soil, cohesion is zero. Angle of internal friction is 32° and unit weight is 110 lb/cft.

For square footing, at shallow depth, according to Terzaghi's formula.

$$\text{qu} = 1{\cdot}3 \text{ CNc} + \gamma \text{DfNq} + 0{\cdot}4 \quad \text{BN}_\gamma$$

For local shear conditions, for $\phi = 32°$
$N'c = 18$, $N^1q = 10$ and $N^1 = 6$

Size of footing : 160 × 180 cm. (5·25 ft. × 5·94).
qu = 110 × 8 × 10 + 0·4 × 110 × 5·25 × 6
= 8,800 + 1,386 = 10,186 lb/sq. ft.
or 4·54 tons/sq.ft.

Correcting for the field load test value, the ultimate bearing capacity of the soil is
y = 0·286 × 4·54 + 0·6,535
= 1·2985 + 0·6535 = 1·951 tons/sq.ft.
Area of the footing = 5·25 × 5·9 = 30·975 sq.ft.
Ultimate load at failure = 1·951 × 30·975
= 60·5 tons.

CALCULATION OF TOTAL SETTLEMENT :

The ' Compression index ' from consolidation test for the soil is 0·265

Intitial ' Void ratio ' is 0·978

$$\text{Total settlement} = \frac{H}{i+e_i}\ Cc.\log_{10}\frac{P_i}{P}$$

Where H is the height of clay layer.

e_i is the initial voids ratio at P.

Cc is the compression index.

P_2 is the initial effective pressure at the middle of the layer

Pi is the final effective pressure.

At the middle of the clay layer over burden pressure.

$P_2 = 12 \times 110 = 1{,}320$ lb/sq.ft.

$= 0{\cdot}645$ kg/cm².

Area of footing = 30·975 sq. ft.

$\frac{r}{z} = \frac{6{\cdot}27}{4} = 1{\cdot}57$, for $\frac{r}{z} = 1{\cdot}57$, NB = 0·02

For 20 tons column load,

$$\therefore \triangle P = \frac{20 \times {\cdot}02}{6{\cdot}27^2} = \frac{0{\cdot}4}{39{\cdot}5} = 0{\cdot}0101 \text{ tons/sq.ft. or kg./cm}^2$$

$$\therefore \text{Pi} = 0{\cdot}645 + 0{\cdot}0101 = 0{\cdot}6551 \text{ kg/cm}^2$$

Since the clay layer below the footing is 8'-0" thick

$$\text{Settlement} = \frac{8 \times 30{\cdot}48 \times 0{\cdot}265}{1+0{\cdot}978} \times \log \frac{0{\cdot}6551}{0{\cdot}645}$$

$$= \frac{243{\cdot}84 \times 0{\cdot}265}{1{\cdot}978} \times \log_{10} 1{\cdot}015$$

$$= 0{\cdot}228 \text{ cm}$$

For the ultimate load of 40 tons.

$$\triangle P = \frac{QNB}{Z^2} = \frac{40 \times 0{\cdot}02}{6{\cdot}27^2}$$

$$= 0{\cdot}0202 \text{ kg/cm}^2$$

$$\text{Settlement} = \frac{243{\cdot}84 \times 0{\cdot}265}{1{\cdot}978} \times \log \frac{0{\cdot}6}{0{\cdot}645}$$

$$= \frac{1{\cdot}228 \times 2{\cdot}03}{0{\cdot}007}$$

$$= 0{\cdot}463 \text{ cm.}$$

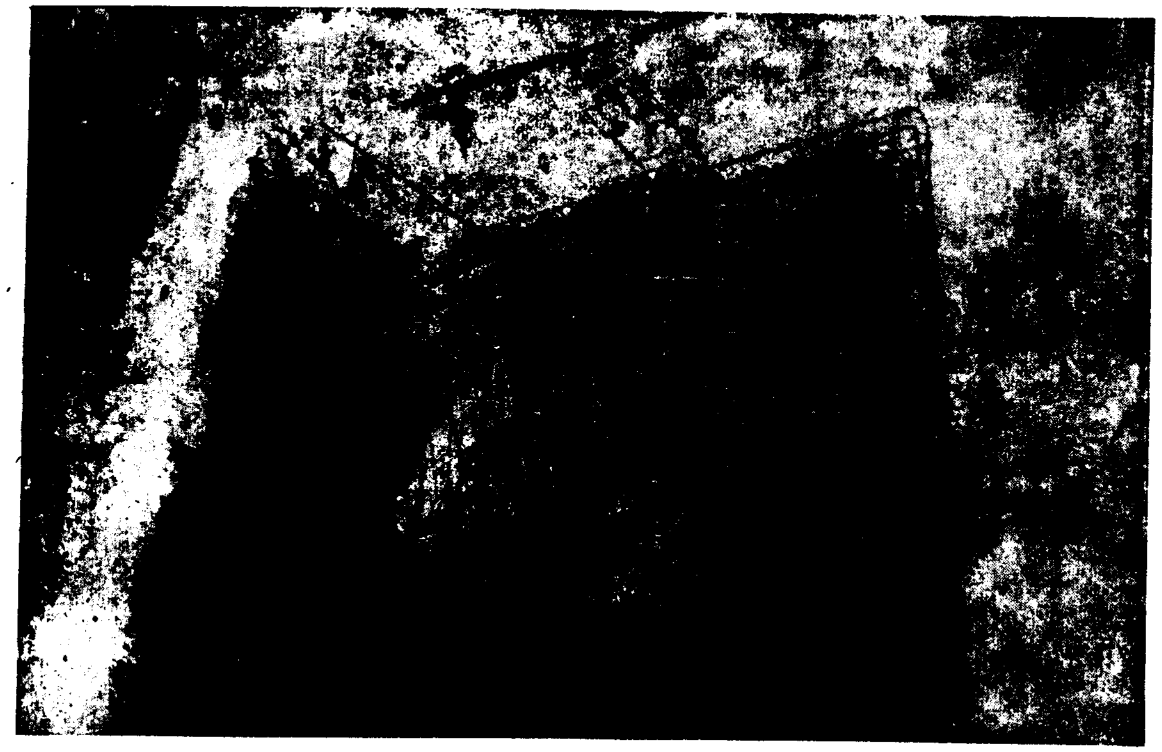
LOAD SETTLEMENT DATA.

Date of test.	*Load in tons.*	*Settlement in mm.*
21st September 1970, A.N.	5	0·157
	10	0·403
	15	0·692
	20	1·190
	22	1·310

Load test discontinued and, rebound noted.

Rebound = 0·41 mm.

Date of test.	*Load in tons.*	*Settlement in mm.*
23rd January 1971 F.N.	10	0·920
	15	1·050
	20	1·200
	25	1·620
	30	2·140
	35	2·940
	40	3·400
	45	3 940



Reinforcement in Position

Test set-up

Test in Progress

Settlement Gauges in Position—A Close-up

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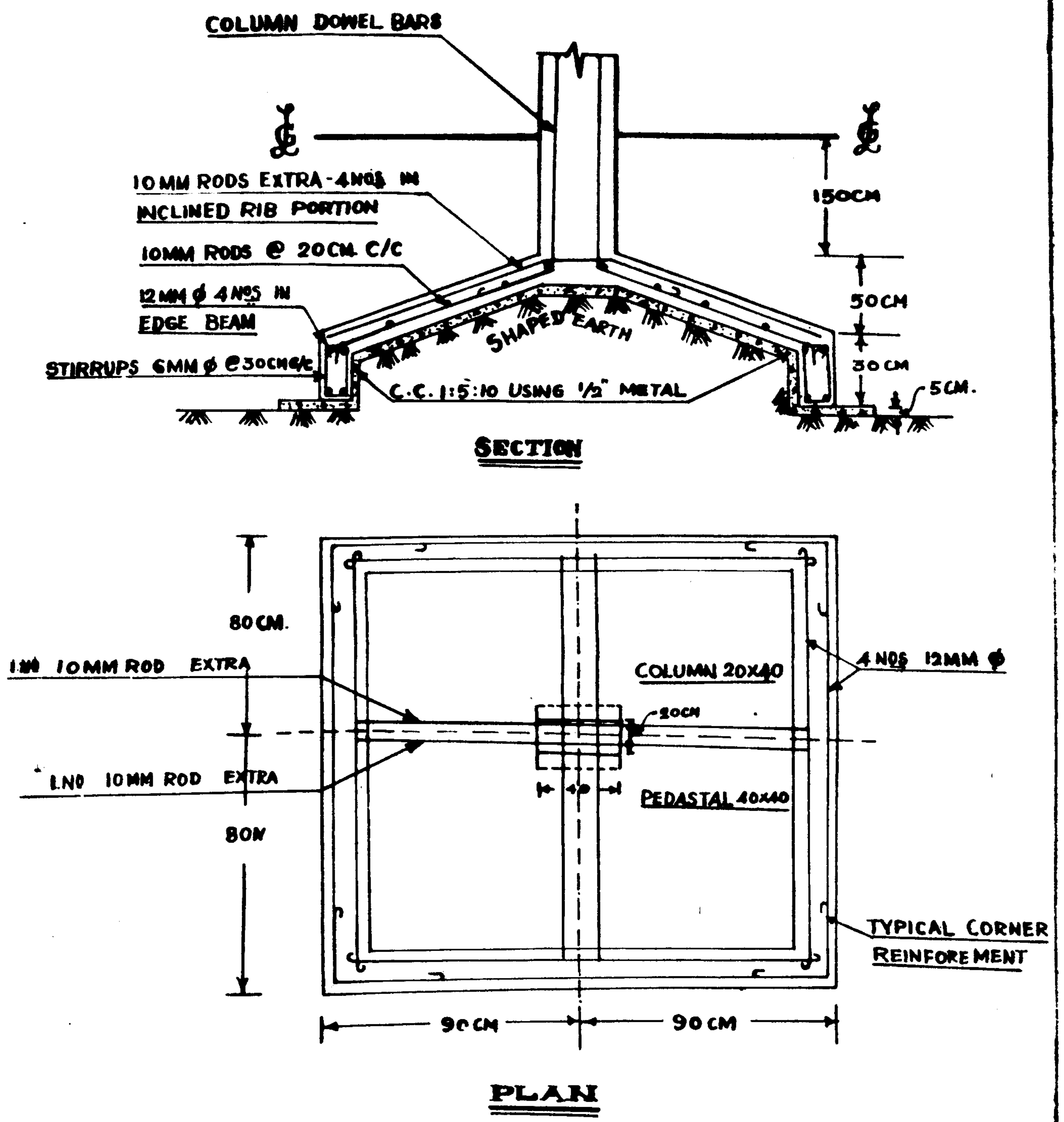
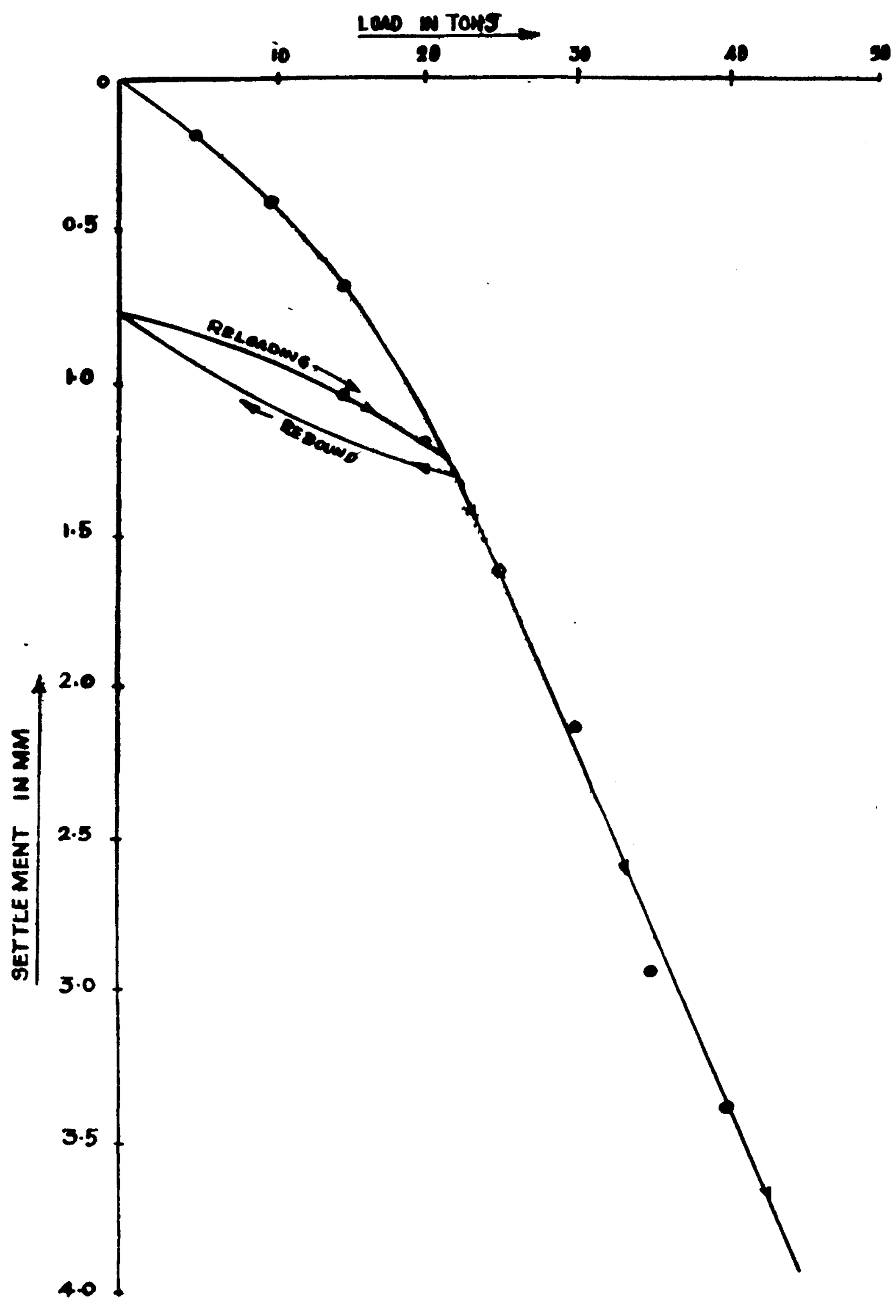
D2m 21,147 N72.13.1



DETAILS OF 'HYPAR' FOOTING (20 TONNES COLUMN LOAD)



LOAD TEST ON HYPAR FOOTING.

GOVT ARTS COLLEGE, RAMNAD.

FLOODS AND DAMS

By

Thiru S. Krishnamurthy, Grade, I.E. (India) *

SUMMARY.

Floods are amongst the most destructive hazards of nature causing enormous damages to crops and properties and heavy toll of human life and cattle lives are also taken. In this paper the usefulness of reservoirs for flood control and other modern methods for mitigating floods are described. Groups of tank systems often found in Tamil Nadu which act as moderators, etc., have also been dealt with.

Floods.

1.1 Floods may be called as high rates of discharges often leading to inundation of lands and are always the result of high rates of surface run off caused by the excess rainfall. Inadequate waterways for allowing free flow of water in rivers during floods result in overflowing of river banks. Increase in flood magnitude is also caused either by the increased intensity of rainfall or by reduced infiltration capacity of soils or in the changed efficiency of drainage system which are impeded by bridges, flood walls, and similar cross masonry structures. Natural calamities like reservoir dam failures, earthquakes and land slides sometimes aggravate the flood problems. In planning flood control structures, the magnitude of flood to be considered for the design is first to be assessed correctly.

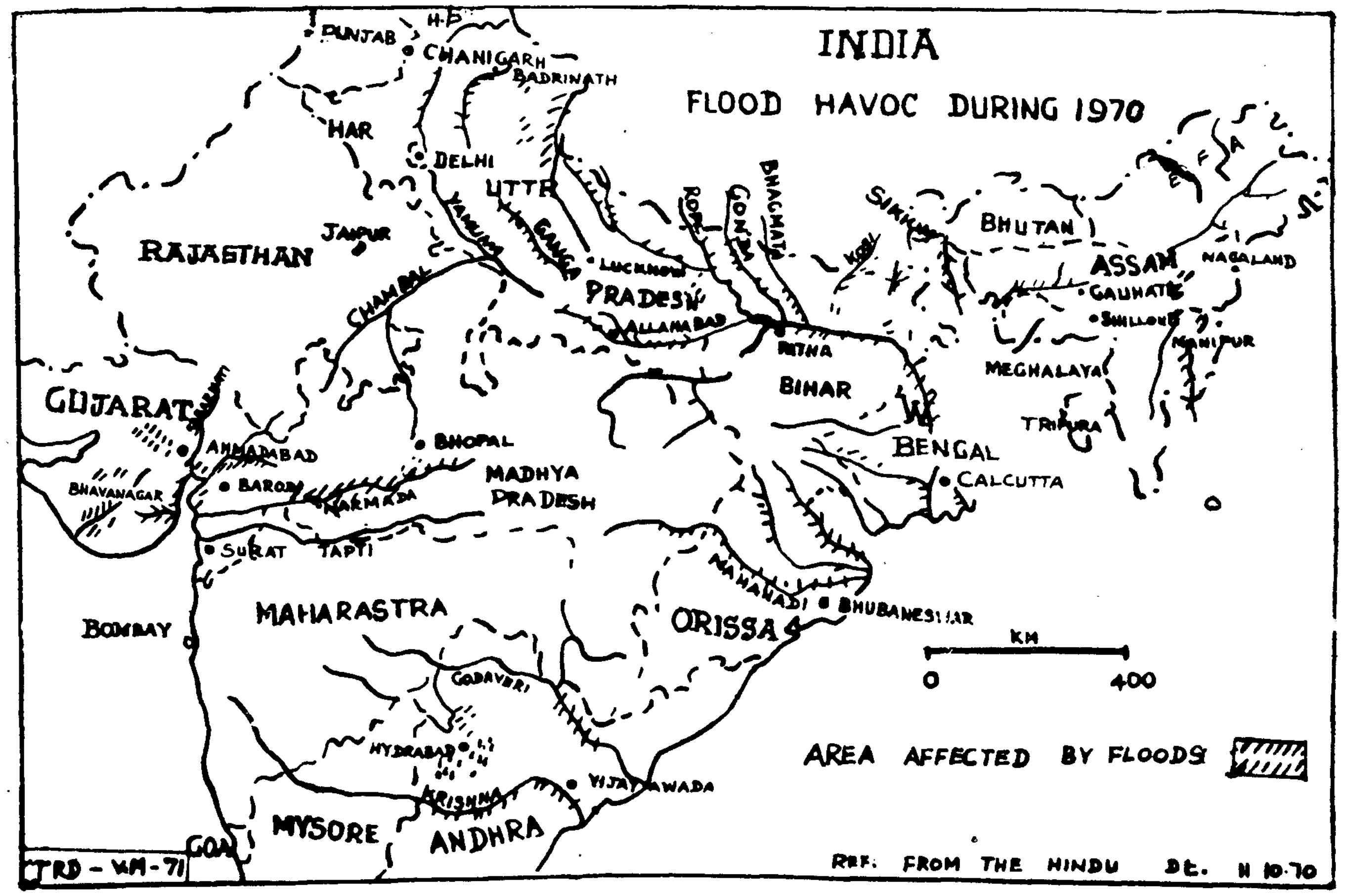
1.2 Some of the various factors influencing the flood magnitudes are:

(*a*) Heavy rainfall in excess infiltration capacity of the soil contributes largely to the surface run off and therefore tends to reach the river rapidly, while a rainfall with lower intensity will be largely absorbed by the soil.

(*b*) When the rainfall duration is equal to or greater than "the mean travel time", the whole catchment will contribute to run off and the run off potential will be much high. As the rainfall duration increases, the infiltration capacity of the soil decreases and larger the rainfall continues at that time, the greater is the resulting flood runoff.

(*c*) A storm moving in the direction of the stream produces a higher peak in a short period than a storm moving upstream.

(*d*) Clay soil or rocky out crops result in the rapid concentration of flood waters.



Flood Ravages.

2.1 Floods have become a serious problems as they cause considerable loss of properties like Railways, Highways, etc., and at times the flood enters green

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lands, villages and towns. Floods also sweep away men and cattle and destroy many houses. An analysis of the figures of the flood damages reveals, that the largest toll namely 70 per cent of the total damages is attributable to crops. The average flood damages in India between the years 1953 to 1965 are as follows :—

Population affected—13.93 millions.

No. of human life lost—512.

No. of cattle lost—26,747.

No. of houses damaged—531,380.

Percentage of affected cropped area to total area damaged—34.4 per cent.

Total cost of damages—Rs. 569.35 millions.

2.2 Disastrous visitations of floods are generally known to be more common on the Nothern and Eastern parts of the country comprising the Ganga and the Brahmaputra river systems than on the Western and Southern parts.

2.3 The following are some of the recorded maximum flood discharges across the rivers in India:

Name of river.	*Anicut site.*	*M.F. discharge.* (IN CUSECS.)
Krishna	Andhra Pradesh at Vijayawada anicut.	11,94,000 on 7th October 1903.
Godavari	Andhra Pradesh at Dowlashwaram.	31,20,000 1953.
Brahmaputra.	Assam	26,00,000.
Gowhathi	Assam	21,90,000 1958.
Kosi	Bihar at Chitra Gorge.	8,00,000.
Narmada	(Year 1968)	20,00,000.
Ganga	Bihar at Ferrukka.	21,25,000.
Mahanandhi	Orissa	15,71,000.
Yamuna	Punjab, Uttar Pradesh.	13,33,000.
Cauvery	K.R.S. Mysore	3,50,000.

The maximum discharge observed so far in the river Cauvery is about 4,00,000 cusecs (observation at Mettur, Tamil Nadu during July 1961). More over the outlay on Flood Control Measures during Five year Plans in Tamil Nadu and Mysore States are 'NIL' as against Rs. 386·8 millions and Rs. 184·45 millions respectively in Bihar and Assam States.

2·4. The appendix No. 1 indicates the flood damages in different States in India from the year 1953 to 1965.

Flood Control.

3.1 The attempts of men in protecting themselves against the floods by one method or on other are as old as the civilisation. Construction of high mounds to escape from the floods and protecting the area by construction of flood bunds and diverting temporarily the courses of rivers during flood times are some of the methods adopted by ancient people.

3.2 A broad indication of the benefits direct and indirect resulting from flood control projects are:—

Direct benefits.—(a) Savings in agricultural, residential, commercial, etc., losses.

(b) Savings in cost of evacuation in flood area, rehabilitation of the affected and building temporary quarters for the evacuators.

(c) Savings in increased cost of transportation charges due to circuitous routes, delays, etc.

(d) Savings in cost of anti-malarial and other health measures.

Indirect benefits.—Higher standard of living due to higher elevation of property and more employment of opportunities by way of development of industries, etc., in the flood protected areas. Some of the modern methods of flood control are.—

Technical measures.—(1) Construction of reservoirs to store flood waters in full or in part.

(2) Construction of flood walls and flood embankment (Kosi embankment scheme is a notable one).

(3) Diverting a portion of flood flows into the adjacent valleys where there is no simultaneous floods.

(4) Improving the river and drainage courses by way of clearing the obstructions.

(5) Soil conservation and afforestation.

Non-technical measures.—(1) Flood forecasting and temporary evacuation.

(2) Flood insurance.

(3) Restricting the occupancy in flood zones.

3.3 In recent years many countries have taken flood control works by various means in the upstream catchment of river basins. In India, up to the end of the Third Five-Year Plan about thirteen million acres of land and 150 towns have been protected from floods. The following are the expenditure incurred on flood control measures :—

	(RUPEES IN MILLION.)
During First Five-Year Plan 1951 to 1956.	132·02
During Second Five-Year Plan 1956 to 1961.	480·58
During Third Five-Year Plan 1961 to 1966.	861·34
Total ..	1,473·94

Flood Control Reservoirs.

4. (i) The purpose of a flood control reservoir is to store the water at times of floods and to release the water after the floods so as to minimise the contribution to the floods in the river in the lower reaches. The reservoir may be constructed for a single purpose of flood control. But where it is economically feasible, it is desirable to utilise the reservoir for the multi-purpose by way of conserving the river flows for power, irrigation, navigation and water supplies. However, the multi-purpose reservoirs designed for irrigation and power have to fulfil the needs of both irrigation and power, whose demand will vary from day to day and seasons to season. Water releases for one benefit often go unused by other and hence these needs are conflicting. Normally under full reservoir conditions, the requirement of water for power generation will be small and for irrigation large heads will not be essential. This aspect has to be carefully examined while designing multi-purpose reservoirs.

4. (ii) The maximum storage capacity of a flood control reservoir is the difference in volume between the design flood and the safe discharging capacity of the river lower down. The reservoir may be large enough to hold the entire flood or number of small reservoirs may be constructed at convenient reaches to moderate the floods.

A study was made in England during 1938 on comparative benefits and cost of large versus small reservoirs on flood reduction shown by thirteen most favourable small reservoirs and by two large reservoirs of equal in total capacity. The drainage area controlled by small reservoirs was only 52 per cent of that of the two large reservoirs. The result showed that the benefit of the two large reservoirs were 12 per cent greater than the system of small reservoirs. The efficiency of a flood control reservoir depends upon the duration of floods. A reservoir may be designed for a particular flood hydrograph. When a flood occurs with the same peak flow but with a larger base, the reservoir might be nearly full or already started surplussing. By the time the peak flow enters into the reservoir there will be hardly any storage space left to accommodate the entire or part of the peak flows and the flood encroaches upon the free boards. Thus maximum potential from a flood control reservoir can be achieved if the reservoir is empty at the time of incidence of floods.

4. (iii) Further the construction of a reservoir results in the reduction of flooding capacities of lower down river courses. By controlled releases from the reservoir, the river in the post-dam periods is no longer subjected to a flood of the same magnitude experienced during pre-dam days. During the stages of high floods, when the reservoir is already full, the reservoir cannot be expected to lessen the intensity of flood flows and flooding lower down portion is inevitable.

4. (iv) The distance between the reservoir site and the area to be safeguarded also plays an important role in effective flood control. Let us assume that a flood discharge of 1,60,000 cusecs passes at the proposed reservoir site. Assuming no further contribution to the river in the lower down, this peak discharge may be reduced due to natural storages in the river to a peak of, say 1,00,000 cusecs by the time it reaches the area to be protected. After the construction of the dam the peak flood of 1,60,000 cusecs may be reduced to a controlled discharge of 40,000 cusecs, which may further be reduced at the area to be protected. Thus the effectiveness of the reservoir as far as the area to be protected is only 60,000 cusecs and not 1,20,000 cusecs. Hence the most effective flood control is obtained from a reservoir of sufficient storage and located just above the area to be protected. Even if the tributory flows are now added in the above example, it will not change the matter much.

5. *Floods and their control in Tamil Nadu.*

In Tamil Nadu the flood problem is not so serious as felt in Andhra Pradesh and other Northern States. The only major river is the Cauvery and there are already two reservoirs built across it one at K. R. S. and the other at Mettur, which mitigate the floods in Cauvery. In other medium rivers such as Bhavani, Amaravathi, Periyar and Vaigai rivers, reservoirs have been built across them during First, Second and Third Five-Year Plans. Salient features of the reservoirs constructed are available at Appendix No. II.

In South India, specially in Tamil Nadu the tanks and storage reservoirs play a greater role in the field of irrigation and flood moderation. These tanks are classified as either 'groups of tanks' consisting of a number of tank receiving supplies from their free catchments and from upper tanks or 'isolated tanks' receiving supplies from free catchment area alone. Such tanks are often found in Chingleput, North Acrot, Tirunelveli, South Arcot and Ramanathapuram districts. These tanks, in addition to being mainly intended for irrigation purposes, often act as good flood moderators especially during early part of rainy seasons when these are practically empty and they store back considerable volume of water which otherwise would increase flood heights. The only problem facing these chain of tanks is that a breach in an upper tank causes series of breaches in the lower down tanks until finally it reaches its outfall. The damages to wheih the lower tanks are subjected to consequent to a breach in upper tank is proportionate to their relative capacities. Strengthening the banks of these tanks, providing adequate flood moderating capacities between FTL and MWL and desilting those tanks should be attended to periodically. Reassessing the flood magnitudes to which these tanks were subjected to, according to the greatest rainfall intensity felt and providing adequate surplussing arrangements to these tanks will considerably prevent such chain of breaches.

Conclusion.

6·1. In a flood control reservoir designed for a multi-purpose the design for free board should, in addition to considering the usual problems like wind velocity, wind set up, wave height and wave up-rush take into account of providing sufficient storage above maximum water level to absorb in full or part of maximum intensity of flood likely to occur when the reservoir is already surplusing. Though such a provision will increase the cost of construction, it will be necessary to control the flood satisfactorily. Such provisions are made in the Hirakud Dam Project across the Mahanadhi river and the dam across the river Damodar. In the case of Hirakud Dam the entire storage of 4·12 m.ac.ft. (Million acre feet) is utilised for flood moderation during South-West monsoon and later on filled for utilisation for power and irrigation.

6·2. A flood forecasting system can be installed at the convenient places of catchment areas so that advance warning can be given and the reservoir level, if already full, can be depleted to receive the incoming floods thus allowing the M.F.L. to raise to a minimum possible. The flood forecasting system are proposed to be set up in Assam, West Bengal, Bihar, Uttar Pradesh, Gujarat and Orissa.

6.3. It is necessary that each river in respect of which floods are a serious problem, should have a a basinwise master plans on the basis of which works should be taken up on a predetermined pattern of priorities. Adequate whole time staff should be engaged to prepare these master plans in all the States. Until this is done the works executed must necessarily be in the nature of adhoc anti-flood measures.

6·4. Present system of flood warning on all inter-state rivers should be improved and perfected. It is most important that during floods reservoirs should be suitably regulated to ensure that the safety of dams is ensured besides absorbing all flood waters.

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APPENDIX NO. I.

Serial number and State.	1959–60 *Gross Cropped area in* 1,000 *Acres.*	*Average annual damage in Rupees in million.*	*Damage in Rs./acre.*
(1)	(2)	(3)	(4)
1 Andhra Pradesh	29,500	5·52	0·19
2 Assam	6,420	43·11	6·95
3 Bihar	27,000	85·01	3·15
4 Bombay *	71,000	3·07	0·04
5 Jammu and Kashmir	1,970	0·72	0·36
6 Kerala	5,670	0·70	0·12
7 Madhya Pradesh	45,000	0·24	0·05
8 Tamil Nadu	17,400	1·58	0·09
9 Mysore	26,100	0·35	0·01
10 Orissa	14,900	44·90	3·00
11 Punjab **	23,900	120·61	5·02
12 Rajasthan	35,700	5·95	0·17
13 Uttar Pradesh	53,600	138·74	2·59
14 West Bengal	15,050	63·06	4·20
15 Delhi	274	4·01	14·70
16 Himachal Pradesh	1,085	0·48	0·44
17 Manipur	225	0·19	0·81
18 Tripura	610	0·83	1·36

* Composite State. ** Undivided.

APPENDIX No. II.

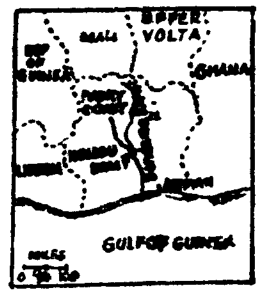
SALIENT FEATURES OF THE MAIN RESERVOIRS IN TAMIL NADU.

Name of Valley and Project.	Location.	Catchment area in square miles.	Gross storage in million Cft.	Ryve's C value C M	Maximum flood discharge in cusecs.	Discharge per square mile of catchment. In CUSECS.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Cauvery—						
Mettur Reservoir	Mettur	16,300	95,660	..	4,56,000	28
Lower Bhavani Project	Bhavanisagar	1,621·90	32,800	725	1,42,750	89
Amaravathi Reservoir Project	Udumalpet	324	4,047	3,000	1,43,400	354
Periyar and Vaigai Valley—						
Vaigai	Periyakulam taluk	870	6,878	690	62,800	73
Periyar	..Travancore	232	15,662	3,500	1,27,410	550
Thambraparani Valley—						
Manimuthar	Tirunelveli district	62·4	5,511*	3,015*	60,000	970
Papanasam	Papanasam	57·5	5,500	3,160*	47,100	820
Kodayar Valley—						
Pechiparai	27 miles from Nagercoil	80	4,350	2,120*	39,000	490
Perunchani	25 miles H. W.	61·57	2,890	2,030*	31,600	514
Ponniar Valley—						
Krishnagiri	Krishnagiri taluk	2,095	3,052	900	1,49,500	71·5
Sathanur	Sathanur	4,180	8,100	965*	2,50,000	60
Varahanadhi—						
Vidur	In vidur	501	605	..	63,030	121

* C. Value derived from max discharge.

. Rockfill Dam is winning race against floods.

Construction is moving full steam on the Ivory Coast's Bandama River Project to build the Kossou Dam high enough in time to meet the summer floods. After almost two years of construction, the $105-million project moved past the halfway mark in February when a rockfill cofferdam closed off the river's flow. The cofferdam will be part of the dam's embankment.



KOSSOU DAM 1-mile-long rockfill is located 186 miles from the coast.

The Project includes Kossou Dam, a 187-ft. high rockfill almost 1 mile long to contain 7 million cu.yd. of fill, and power transmission lines to the coast. The reservoir, now being filled, will cover 650 sq. miles and store 24,000 acre-ft. for irrigation, water-supply, recreation and navigation. The project is intended to help the Ivory Coast double its total energy capabilities, meet energy demands through 1980 and spur a regional economic upswing.

The project is located on the Bandama Blanc River, 10 miles from its confluence with the Bandama Rouge River and 186 miles from the capital city of Abidjan on Africa's west coast.

The dam's intake structure, adjacent to the left abutment, is designed for hydraulic, wheel-mounted gates located within the trashrack structure.

The 23-ft.-dia. power tunnels will be steel lined and from 354 ft. to 384 ft. long. No surge tanks or butterfly valves will be necessary. A chute spillway cut through the left abutment and regulated by three radial gates, will handle 81,000 cfs.

Cofferdam design changes.—Originally, the cofferdam was to be built in two stages. First, a horseshoe-shaped cofferdam was to be extended out from the left embankment to just short of a diversion channel cut parallel to the main stream. Then, on the upstream side, an extension was to cut off the river's flow. Engineers decided to run the cofferdam from both toes of the embankment instead, building up the embankment behind the cofferdam. In the process, the cofferdam is being incorporated into the dam's embankment.

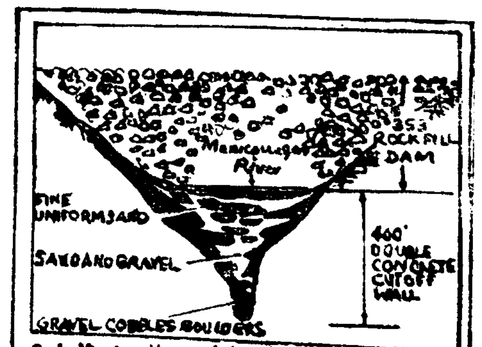
Work on the dam is co-ordinated around the river's July to mid November flood season. During the first stage of the project, the diversion channel was excavated to handle the river's flow while the cofferdam was being built over the riverbed. With the cofferdam now extending over the entire river, a temporary low-level outlet is handling the flow. With the level of the reservoir expected to rise rapidly during the flood season, the embankment height must be kept safely ahead. *By the end of August, engineers hope* to have the dam above the reaches of the expected flood flows. Crews also expect to complete the spillway in August in time to handle the flood flows. Almost all of the rockfill for the cofferdam and the embankment will come from the spillway excavation.

The switchyard is situated downstream of the power-plant between the tailrace and spillway channels. Current power generation of the Ivory Coast is 110,500 kw. Kossou Dam will more than double this. Half of the electricity is now generated at two hydro-electric plants, and the rest at various gas turbine, stream and diesal units in and around the larger cities. Officials expect Kossou on line in September 1972.

Engineering News-Record, May 13, 1971.

b. Deepest cut-off begins at Manic 3 Dam site :

Hydro-Quebec's dam builders are at it again, trying something that's never been done before. Work began this month on the concrete cut-off wall at the Manic 3 dam site on the Manicouagan River, 75 miles downstream from the World's highest multiple arch dam, where Manic 5 is now on line.



Cutoff goes through boulder laced alluvium.

This 2-ft.-thick pile and panel concrete membrane (actually, there will be two parallel walls 8 ft. apart) will extend 400 ft. deep to plug the alluvium filled, V-shaped gorge (*see drawing*).

Icanda Corp, (1969) Ltd., of Montreal, specialists in deep slurry-wall cutoffs, will be going deeper than they, or anyone else has ever attempted to seal the foundation for the 353-ft.high earthfill dam. Manic 3's undergorund power-house will have a capacity of just under 1.2 million kw.

The alluvium Icanda must block off will make the job challenging. The gorge is filled with deposits of sand, gravel, cobbles and boulders up to 5 ft. in diameter. Hydro-Quebec specifications call for a maximum deviation from vertical of 6 in. for both piles and panels at any depth.

The twin cut-off walls will have interlooking 24-in. piles in their center sections where the gorge is deepest, with panels 2 ft. wide and 11 ft. long forming the shallower flanks. Two crosswalls will link the cut-offs at the points where the piles and panels meet.

Icanda uses bentonite slurry to stabilize the walls and prevent cave-ins as the pile holes and panel trenches are excavated. Tremied concrete displaces the slurry to create the cut-off elements.

Manic 3's 55-ft.-dia. diversion tunnel is under construction and must be completed by this November to allow cut-off construction in the riverbed.

Engineering News-Record, April 22, 1971.

c. ***Highest U.K. Dam for Llyn Brianne :***

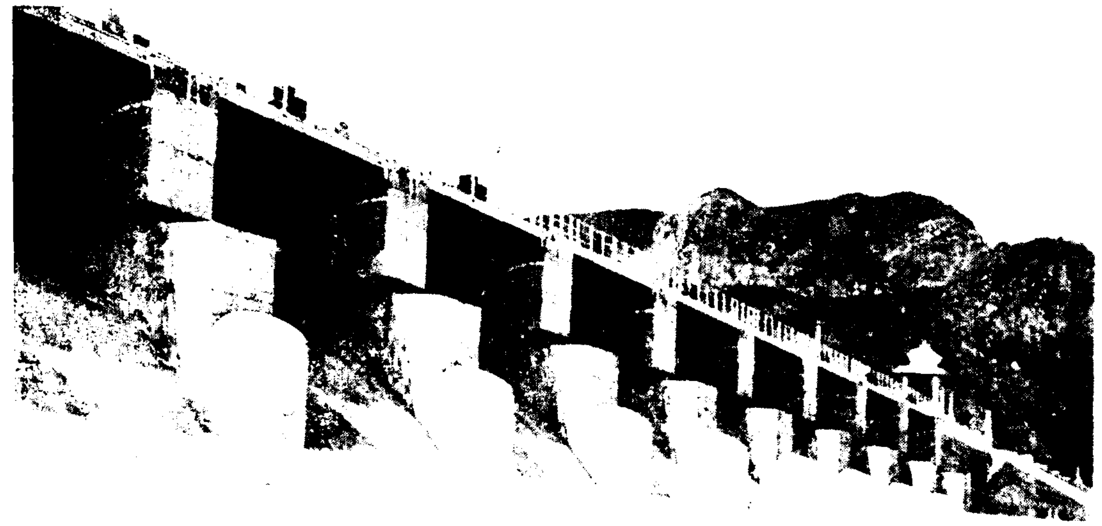
Severe water shortage in South Wales, which has caused rationing in recent years, has led to the construction of the highest dam in the British Isles. The dam, which is part of the River Thwy Scheme, is being built at Llyn Brainne in Carmarthenshire.

The location of the dam set in a narrow gorge high in the welsh Hills, is particularly suitable because the valley to be floode l contains no arable land or occupied houses. The project, was started in 1968 and is scheduled to finish in 1972. When completed the dam will be 91 m. high, with a crest length of 270 m. and a base width of 335 m. Obviously, it is economic if local materials are used in the construction. Thus the watertight core of the dam will be of clay, which is being hauled from a clay pit 12 km. from the dam; supporting the core on either side will be shoulders of rock blasted from the abutments to the east of the dam and from the spillway. Between the rock fill and the core will be transition zones of crushed rock and beneath the downstream shoulder, there is a free draining layer or rock. The upstream face of the dam is protected against wave erosion by a rip-rap layer of larger rock.

Before any work could be started on the construction of the dam a diversion tunnel had to be built to divert the Thwy throughout this period of construction. A tunnel 450 m. long and 4 m. dia was driven and fully lined with concrete. Some 970 grout holes had to be drilled and grouted to fill cavities in the rock. 200 drainage holes were then drilled to prevent a build up of water pressure around the tunnel. For this extensive drilling, Thyseen, used Atlas Copco Puma hand-held rock drills.

When the reservoir is complete Llyn Brianne will have a surface area of 210 hectares and will drain catchment area of 88 km². with an average rainfall of 1,800 mm. The dam is expected to be fully operational in the first winter months of its completion but it will be capable of meeting the growing water demands of Caramarthenhire and West Glamorgan for another 30 years. The estimated cost of the whole River Towy scheme is £12.7M.

The Consulting Engineer, August **1971.**



ALIYAR DAM SPILLWAY