[Ex/66]

UNIVERSITY OF ROORKEE,
ROORKEE (U.P.)

Certified that the attached Thesis Dissertation on " Construction control of High Earth Dams ."

was submitted by

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and accepted for the award of Degree of Doctor of Philosophy Master of Engineering in

W.R.D.

vide Notification No. Ex/146/P-65(Degree)

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Dated Dec. 20, 1967

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CONSTRUCTION CONTROL OF HIGH EARTH DAMS

A Dissertation
submitted in partial fulfilment
of the requirements for the Degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

by

ATUL KUMAR VARMA

WATER RESOURCES DEVELOPMENT TRAINING CENTRE
UNIVERSITY OF ROORKEE,
ROORKEE.
October. 1967

CERTIFICATE

Certified that the Dissertation entitled "Construction Control of High Earth Dams" which is being submitted by Shri Atul Kumar Verma in partial fulfilment of the requirements to Degree of Master of Engineering in Water Resources Development of University of Roorkee, is a record of student's work carried out by him under my supervision and guidance. The matter embodied in this has not been submitted for any other Degree or Diploma.

This is further to certify that he has worked for a period of over 12 months from 1.10.66 to 17.10.66 in connection with the preparation of this Dissertation.

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Dated:- The 19th Oct '67.

CERTIFICATE

Certified that the dissertation entitled " Construction Control of High Earth Dams" which is being submitted by Shri A.K. Verma, in partial fulfilment of requirement of M.E.(WRD) of the University of Roorkee is a record of the student's work carried out by him under my supervision and guidance.

This is further to certify that he has worked for a period of 10 months from November 1966 to August 1967 in connection with the preparation of this dissertation.

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Tonughat

Dated:- The 9th Sept, 1967

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S Y N O P S I S

Construction of High Earth Dams in recent years has been on increase, necessitating development of quick control methods to keep pace with the construction and yet not endangering the safety of structures. This could be achieved by proper understanding of the subject and employing rapid control techniques which may not be rigorous but must give results sufficiently accurate for all practical purposes. Different methods for such control are available but all of them have their own limitations, merits and demerits. A review of all such methods is therefore necessary before choosing one or the other according to one's own requirement and for achieving best results. Secondly, in a big project consisting of various activities, there is need to have better coordination between all such activities and utilising the resources at command in the best possible manner. This is possible by a good construction schedule, able to foresee all works and handicaps in advance and suggest remedial measures in the latter case. There are various methods for devising such control also which again have their own usefulness for one purpose or the other. A review of such Control Techniques also is, therefore, necessary in context of constructing high earth dams and their control.

Above two points form the main nerves of this Dissertation where numerous illustrations of Earth Dams failure and field investigations have also been brought in to elucidate the points. Certain definite steps have been also suggested in the concluding remarks, which will be helpful in accelerating pace of high earth dam construction.

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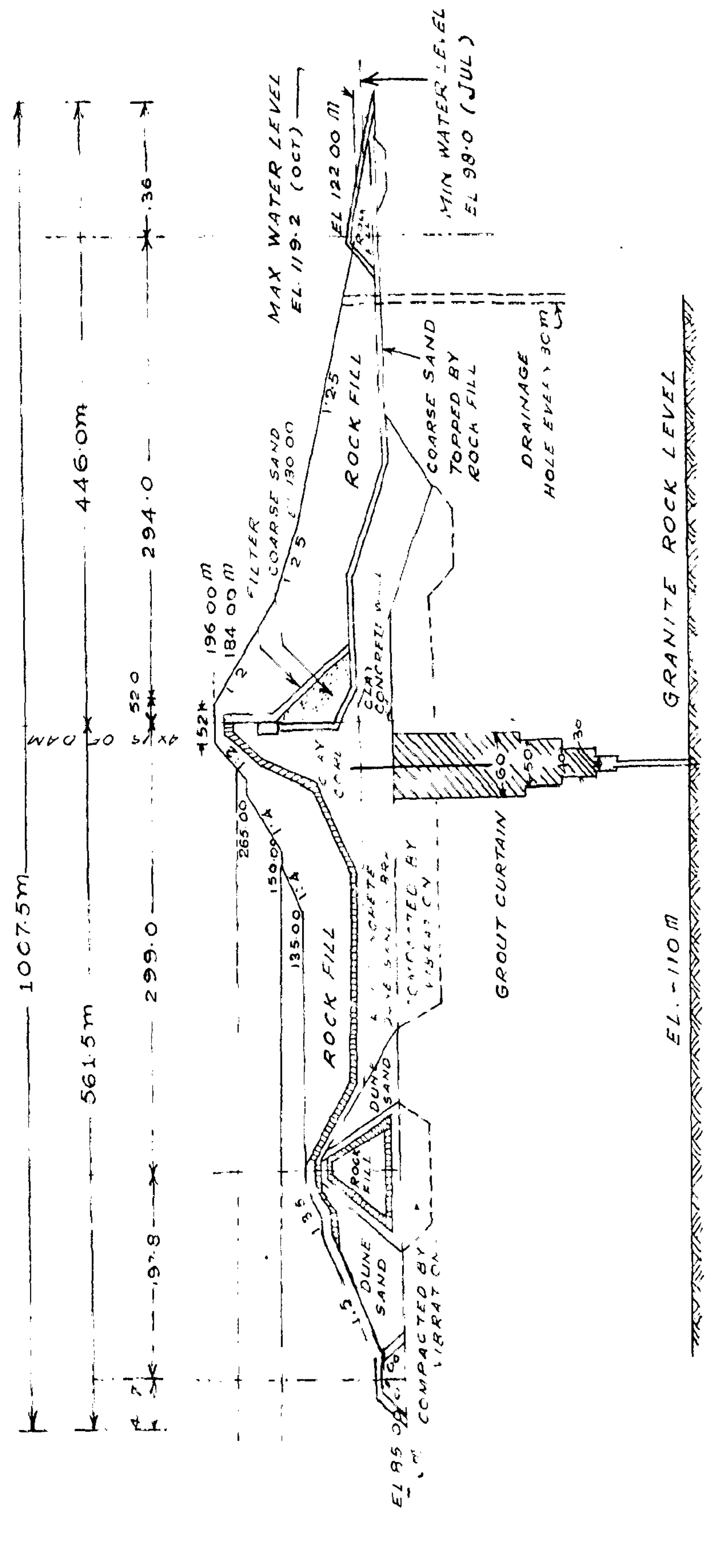
CHAPTER - 1

INTRODUCTION

'Construction Control' of High Earth Dams is different from that of ordinary embankments, in that the former requires a much more basic understanding of Soil Mechanics principles and their application to Earth Dams. Ordinarily an earth dam exceeding 50' height will warrant all such precautions as necessary for High Earth Dam Construction, their construction control being primarily dependent upon type of construction and the problems unique to the dam site. Besides, a general control is also required to be exercised common to all similar type of dams. Such control includes quality tests for different materials being used on the dam, in general, and the routine tests for density and water content of core and shell material, in particular. It also requires that measures are taken to complete the works as per plans and specifications, unless unavoidable situation warrants to the contrary. Special control includes safety measures to be taken against construction of earth dams in earthquake regions, construction pore pressures and likely loss of water due to excessive seepage etc. A recent trend is to construct earth dams, higher and higher and thus economise the cost of construction. Some noted examples of highest earth dams are Nurek Dam, 990' (U.S.S.R.), Oroville Dam, 700' (U.S.A.) and High Aswan Dam, 360' (U.A.R.) A cross section of the much publicised High Aswan Dam is shown at Fig.1.1. It will be clear from the figure how the sections have been changing from U/S face to D/S face. Apart from such unusual sections, presenting great difficulties during construction, designers and theoreticians have to ponder a lot and do good amount of mental exercise before a section is finally decided.



HIGH ASWAN DAM
(NOT TO SCALE)
FIG. 1.1

It is only recent advances in Soil Mechanics that have made it possible too to go in for such higher earth dams because forces on flexible soil mass can now be better analysed in Earth Dams designs than it was possible two decades past. Still, this is a fact that soil properties show a wide range of variation from State to State, region to region and even in one pit of the same borrow area. In that context, it becomes obligatory for a Construction Engineer to ensure that soil being used in dam construction have the same properties (may be within tolerable limits) which have been taken in the designs of earth dams structures. Second responsibility which has descended from the construction of high earth dams is making dam safe against rapid construction i.e. to complete the structure as early as possible without dispairing quality of work to an extent which would endanger its safety. It has been noticed in the past that accelerating construction has resulted in failure of some of the earth dams, even while construction. Their causes have been investigated in context of pore pressure, relatively a new theory of Soil Mechanics, and counter-measures have been suggested and employed on a good many dams to success.

Similarly, various compaction implements are today available in the market and there is need to review all such implements, used earlier with their derived results, for their economical deployment on new high earth dams to be built. Not very long ago, founding earth dams on deep pervious foundations used to pose serious construction problems. New techniques of alluvium grouting and concrete diaphragm has made the task easier. Problems of liquefaction of fine sand are being studied in context of dynamic loading by creating explosive blasts to simulate earthquake conditions. Such tests help in ascertaining the effect of earthquakes on pervious foundation, and in one way it causes a relief to Construction Engineer, in that he is having no more nightmare about results of construction, careful though cropped with uncertainties till actual commissioning of reservoir. By such site test, it can be ascertained whether an extra measures (of 'control') in certain cases are warranted or not. For example, if river bed sand does not liquefy under simulated earthquakes conditions of design magnitude, there is no need of further compacting the sand without any economical gains.

Though the construction control of high earth dams covers quite a wide range of activities for construction engineers, only some of them which are quite important for dam construction have been dealt with in subsequent Chapters, in particular context of rolled fill construction. Emphasis has also been laid on how to economise the construction-time by exercising time-control and rapid construction control methods. Scope of construction control broadly covers following points.

(1) Time Control

(2) Laboratory & Field Investigations

(3) Foundation Treatment

(4) Embankment Construction, and

(5) Instrumentation

While first aspect of control is separately dealt with at Chapter [illegible], other aspect of control has been covered by Chapter [illegible] on on "Field Control". Subsequent Chapters have been needed to further explain certain important points of construction control in detail. Besides, a set of earth dam failures have been discussed in Chapter 8 to show how construction lapses can be eliminated with a more understanding of the subject and by being more careful during construction. Concluding remarks have been given in the last Chapter with illustrations from Tenughat Dam Project now in advance stage of construction. This will help in deciding practical adaptability of different controls to be exercised during actual construction of High Earth Dams (Rolled fill).

CHAPTER - 2

TIME CONTROL

2.1. PLANNING AND CONTROL TECHNIQUES

Relatively important and conventional planning and control methods in vogue are:-

(i) Gantt Chart

(ii) Milestone Chart

(iii) Flow Process Chart

(iv) Line of Balance Chart

2.1.1. GANTT CHART (Fig.2.1)

Developed by Henry Gantt (U.S.A.) prior to World War I this Chart is primarily meant to display a work schedule by means of the bars drawn on a time scale (Fig.2.1). According to Gantt, the bars could be shaded from time to time to indicate progress also. The bar or Gantt Chart System is quite simple and can be easily understood. The main deficiencies are:-

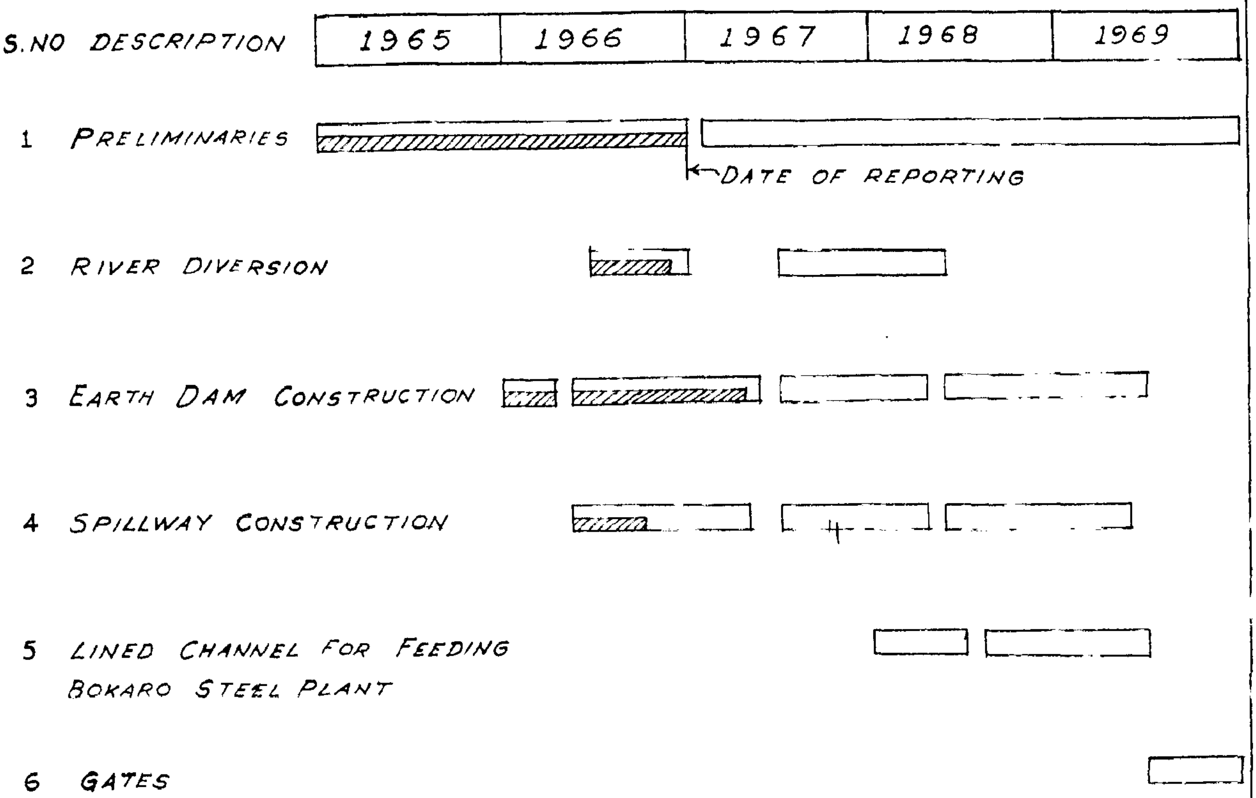
(a) It does not indicate the effect of some work being behind or ahead of schedule on the project completion time or the effect of delay on other items of work.

(b) Reporting of progress involves estimating very broadly as to how much of total work is completed, reason being that the Gantt Chart does not show any milestones or events between the horizontal bars.

2.1.2. MILESTONE CHART (Fig.2.2)

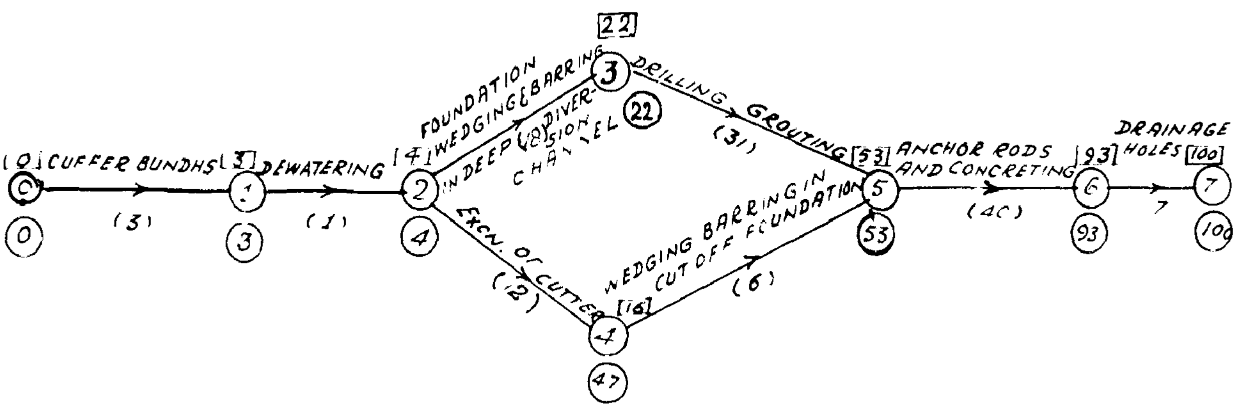
Milestone chart, an improvement over the then existing Gantt Chart was developed around 1940. It differs from Gantt Chart in that it has introduced the concept of events or intermediate

CONSTRUCTION SCHEDULE FOR TENUGHAT DAM PROJECT



GANTT CHART

FIG. 2.1

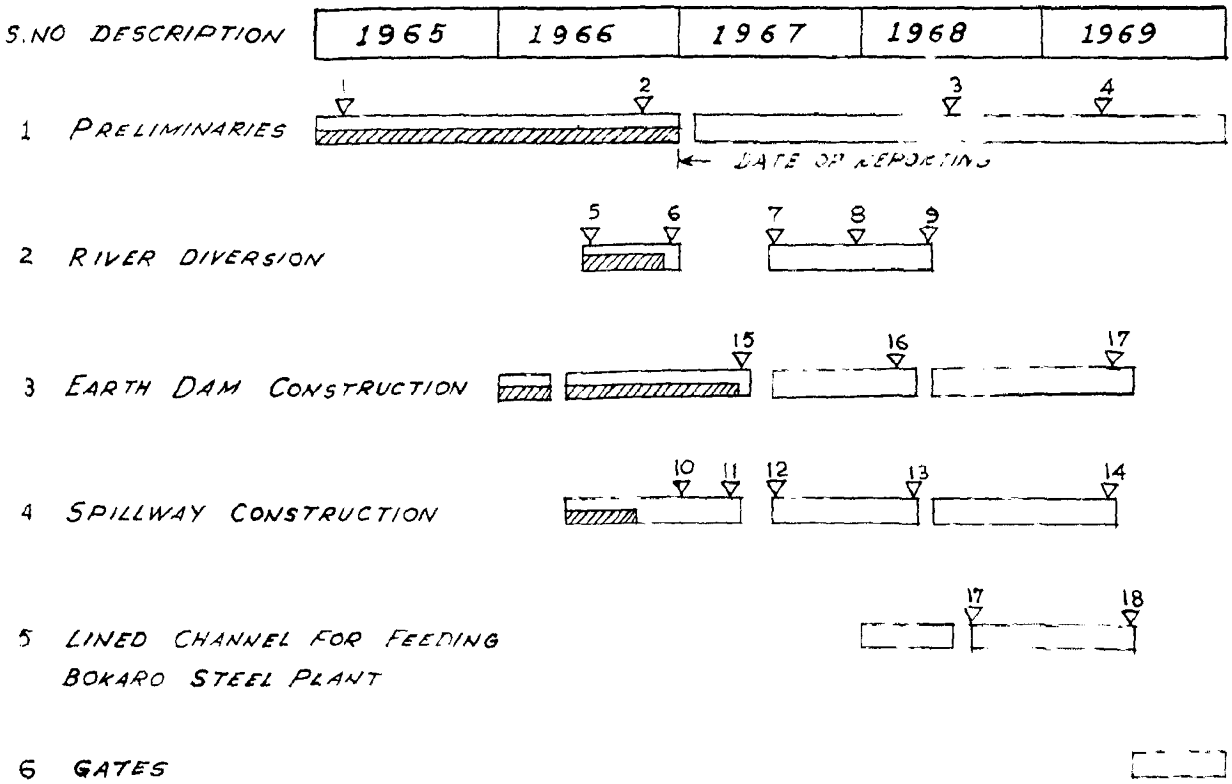


C.P.M SCHEDULE FOR CONSTRUCTION SLUICE

FIG. 2.7.

CONSTRUCTION SCHEDULE FOR
TENUGHAT DAM PROJECT

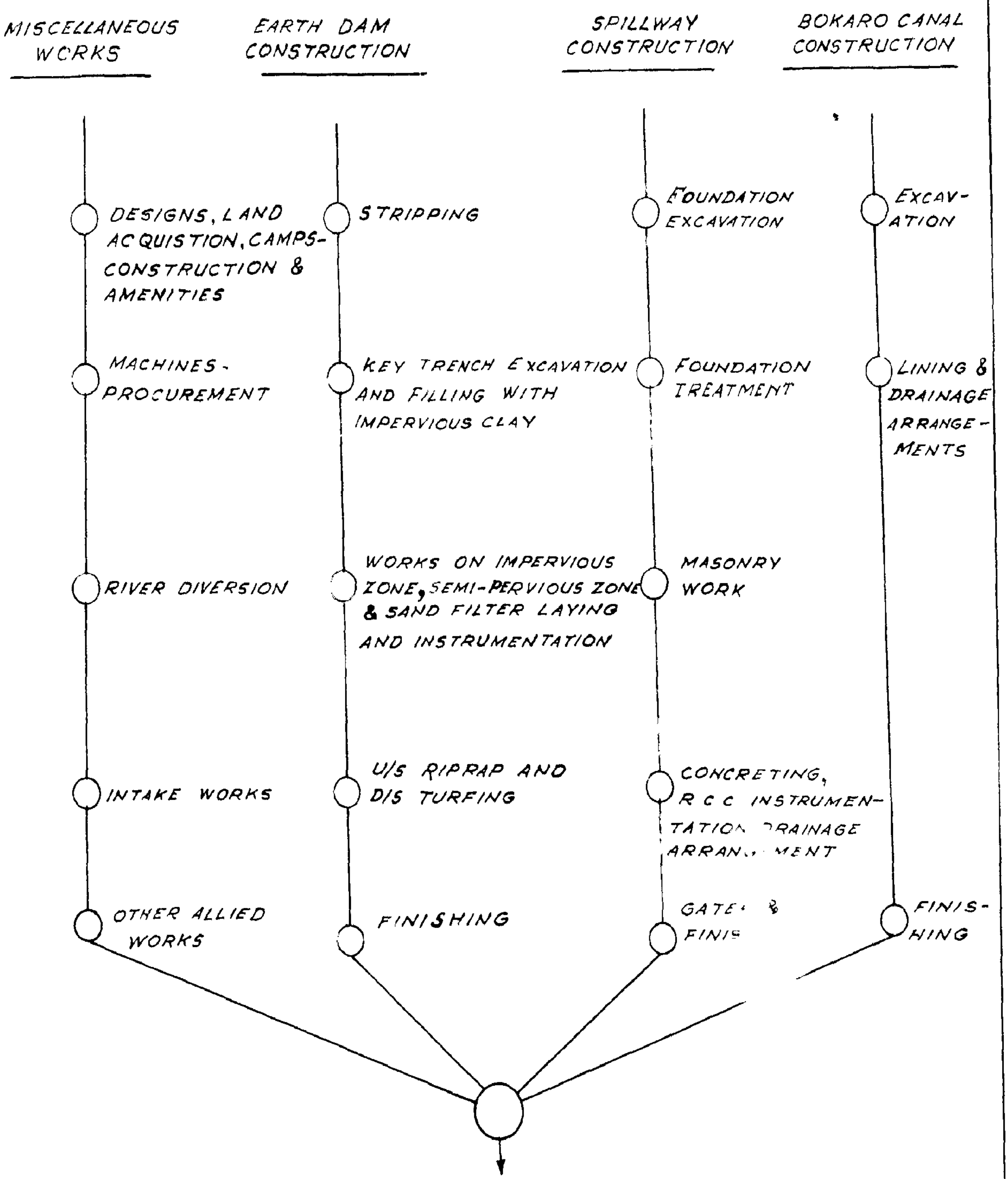
FIG. 2.2



MILE STONE CHART
FIG. 2.2

MILESTONES (∇)

(1) DESIGNS FINALISED (2) MACHINES PROCURED (3) BOKARO CANAL ALIGNMENT FINALISED (4) TAKING OF INVESTIGATION FOR SECOND STAGE (5) START EXCAVATION OF DIVERSION CHANNEL AND DEEP DIVERSION CHANNEL (6) COMPLETE DEEP CHANNEL AND CONFER DAM (RIVER DIVERTED THROUGH DEEP CHANNEL). (7) START CONSTRUCTION SLUICE AND EXCAVATION OF DIVERSION CHANNEL (8) RIVER DIVERTED THROUGH CONSTRUCTION SLUICE (9) RIVER DIVERTED OVER PARTLY CONSTRUCTED SPILLWAY (10) COMPLETE FOUNDATION EXCAVATION OF SPILLWAY (11) FOUNDATION TREATMENT COMPLETED (12) START MASONRY IN OVERFLOW AND NON OVERFLOW BLOCKS (13) COMPLETE MASONRY UPTO R.L. [illegible].00 (14) COMPLETE MASONRY UPTO R.L. 872.00 (15) COMPLETE CONCRETE CUTOFF IN RIVER BED (16) EARTH DAM RAISED TO R.L. 872.00 (17) START CANAL LINING (18) START SUPPLYING WATER TO BOKARO STEEL PROJECT



WATER SUPPLY TO BOKARO STEEL PLANT

FLOW CHART

FIG. 2.3

milestones which are inserted along the task or activity oriented horizontal bars as shown in Fig.2.2. Quite evidently, because of the intermediate milestones, progress reporting can be more definite by this method. It again lacks inter relationship, and interdependence among different events or tasks and as such it cannot predict whether a project would lag or lead in its complete period, as a result of schedule slippage during various stages of the project construction.

2.1.3. FLOW PROCESS CHART (Fig.2.3)

These charts were developed by the Gilberths the contemporaries of Gantt, and they exhibit "sequential relationship of activities" or "inter-relationship between tasks" (Fig.2.3).

However, the flow process charts do not indicate time duration of individual activities and as such do not help in predicting the completion time of the project.

2.1.4. LINE OF BALANCE CHART (Fig.2.4)

Line of balance chart is a combination of the flow process chart and Gantt chart where the tasks are plotted or a time scale according to their sequential and lead time relationship leading to the final event, as shown in Fig.2.4.

2.1.5. NET WORK TECHNIQUE (Fig.2.5)

This technique of project scheduling and progress reporting was developed under variety of names, principal of them, being

(i) Critical Path Method (C.P.M.)

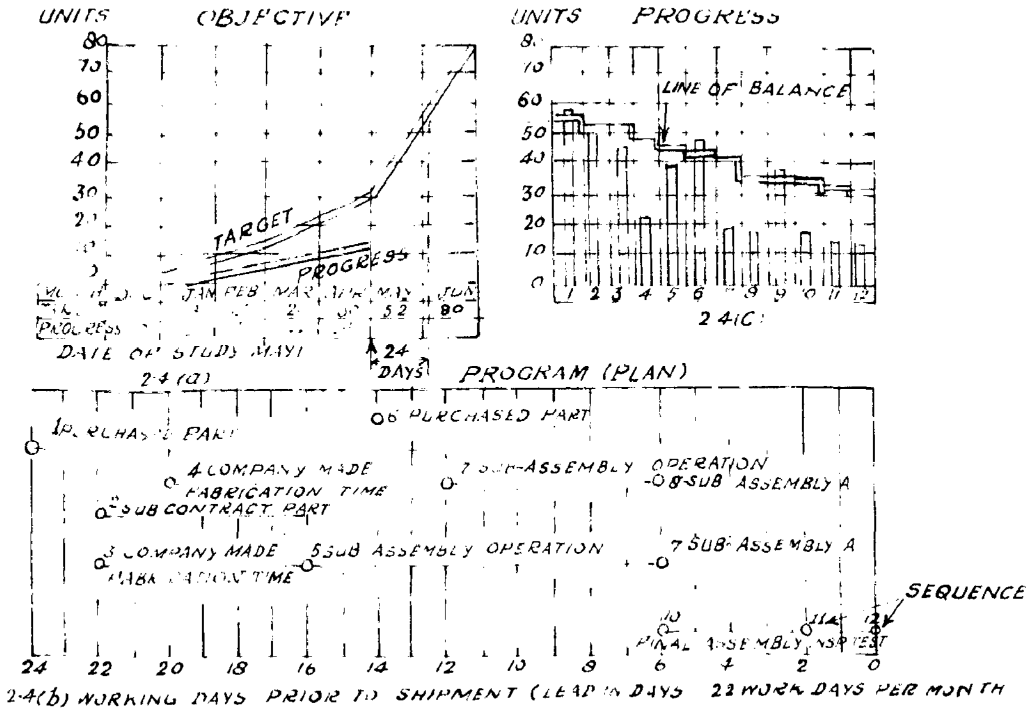
(ii) Programme Elevation and Review Technique (PERT)

(iii) Resources Allocation of Multipurpose Scheduling (RAMP)

They are briefly discussed below:-

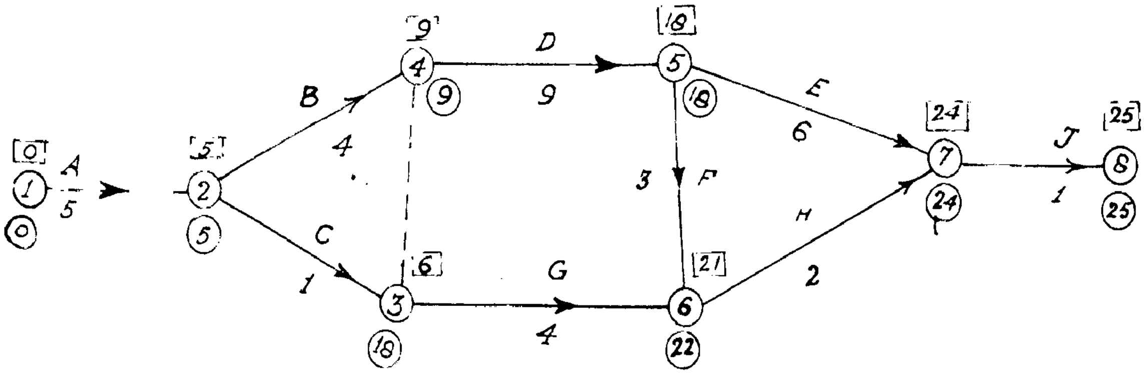
(i) CRITICAL PATH METHOD

This method was first developed in year 1957 by Morgan R. Walker of E.I. Du Port and James Kelly of Remington Rand in U.S.A.



LINE OF BALANCE CHART PRODUCTION PROTOTYPE.

FIG. 2·4.



NETWORK OF A PROJECT

FIG. 2·5

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Du Pont had earlier experienced considerable losses because of the stoppage of production during regular overhaul and maintenance of the some of their vital plants. When C.P.M. was applied during overhaul of the plant, amazingly enough the shut down period was reduced from 125 to 93 hours.

(ii) PROGRAMME EVALUATION AND REVIEW TECHNIQUE (PERT)

This method was evolved in 1958 by the U.S. Navy's special project office and a management consultant firm, Booz Allen and Hamilton to provide for the integrated planning and control of the polaris missile (weapon system) projects. PERT is credited for having reduced completion time of the above project by as much as two years, and at the same it commanded a good coordination between works by different agencies.

Though, above two methods were independently discovered, they were more or less similar. Now-a-days they are known as "Network Technique". Principal difference between above two methods is that CPM networks are "activity oriented" while PERT are "event oriented".

Some examples of events and activities are given below-

(a) Events are:-

Contract awarded

Project approved

Foreign exchange sanctioned

System tested

(b) Activities are:-

Preparation of designs

Scrutiny -- of tenders

Laying pipes

Equipment erection

Processing etc.

(iii) RESOURCE ALLOCATION OF MULTIPURPOSE PROJECT SCHEDULING (RAMPS)

This is latest technique, able to work for many projects simultaneously, with different completion dates but calling on the same man power, materials and equipment resources. Because, we are considering construction control of a single earth dam project here, we would not build up more on it hereinafter. This also requires use of computor events and activity being large.

2.2. CONTROL TECHNIQUES AS APPLIED TO A CONSTRUCTION PROJECT

Developing a network and ground rules for net work development are listed extensively in the papers by C.S. Parthasarthy[1] and V.G. Gokhale[2,3,4]. Author I would discuss here only how Network technique applied to an earth dam project (Tonughat Dam Project for example) compares with other conventional methods (Vide Fig.2.1 to 2.4). Critical path method is discussed in greater details in para 2.3.

2.2.1. GANTT CHART

Gantt Chart, alternatively known as Bar Chart is quite simple and still widely used all over the world. Six broad items of Tonughat Dam Project represented on this chart in Fig.2.1 show that on the date of progress reporting, preliminary works are on time and earth dam construction are ahead of schedule while River Diversion works and spillway construction lag behind. It cannot indicate how the project as a whole is going to lag or lead the completion period, as a result of individual description of works being late or leading on a particular date. From the illustration, it cannot be established whether the delay in spillway works will be compensated by the earth dam construction being advance of schedule. And this is attributed to the reason that the chart does not contain any inter-dependence or relationship between different events or activities. Also, reporting of progress by hatched lines is approximate in so far as it only indicates that the work was in progress on the date of reporting and not complete.

2.3.2. MILESTONE CHART

Since milestones (∇) indicated by numbers are introduced on the above Bar Chart (or Gantt Chart) also reporting of progress can be assured to be more definite. For examples, event($\overset{6}{\nabla}$)(River Diversion Through Deep Diversion Channel) could not be completed in time. Event ($\overset{15}{\nabla}$) (concrete cut off construction in river bed) may still have chance to be completed in time provided remaining 10 % (approximately) work of river diversion is pushed up. Event ($\overset{2}{\nabla}$) i.e. procurement of machines have been satisfactorily done etc. etc. This chart also lacks dependencies and inter-relationship between different tasks and activities and hence again it cannot indicate whether Tenughat Dam works would be completed in 1969 or delayed beyond that as a result of schedule slippage in one of the milestones during various stages of the project construction. However, this milestone chart can be preferred over Gantt chart for more definite progress reporting.

2.3.3. FLOW PROCESS CHART

This flow chart of Tenughat Project shows graphic and sequential relationship between different descriptions of works of the project. It is useful for a planner to have clear picture of works in the fashion shown in Fig.2.3 as this shows inter-dependence between various activities, and contains events (or milestones) as well. However, because this does not indicate time duration of individual activities, its utility in determining time schedule is only little. This is not very useful for stage progress reporting also.

2.3.4. LINE OF BALANCE CHART

Usually useful for repetitive operations, it is not very useful for non-repetitive operations like construction projects or research and development projects. This chart will not, therefore, be discussed in the context of Tungabhadra Project.

2.3.5. NETWORK TECHNIQUE

No matter what the size, the type of job best suited to the application of this technique is one which has a single objective and is undertaken only once, that is, involving non-repetitive processes. C.P.M. is specially useful in planning scheduling and of control of construction projects, research and development, overhaul and maintenance programmes, weapon system development, new product development etc. The technique can be introduced in any phase of a job. However, its full potential is realised best when the technique is used from the initial stages of inception and through the life of the project.(1)

Before applying this technique to a construction project, following terminologies should better be understood.

(1) ACTIVITY ELAPSED TIME (t_e) is activity duration i.e. time required for the performance of an activity assuming application of normally available and expected resources as planned such as man power, capital etc.

(2) EARLIEST EXPECTED EVENT TIME (T_E) is the earliest time when an event may be expected to occur and T_E for an event is calculated from the beginning event by accumulating or summing up the activity time through the longest path on the network (critical path). Where two or more activities constrain a single event, T_E is calculated along each

path and the longest time is chosen as the T_E of the given event. This is based on the principle that an event can occur only after all the activities preceding it are completed. T_E is written in a rectangular box above the event as shown in Fig.2.9. T_E for the end event determines the total project duration, i.e. the time required to complete the whole project.

(3) <u>LATEST ALLOWABLE EVENT TIME</u> (T_L) is the latest time when an event can occur without creating any expected delay in the completion of the total job i.e. the end event. The T_L for the end event is set as equal to the T_E of the end event or equal to any predetermined specified or directed time. The T_L for any event "X" written in circles below the events is then calculated backward from the end event as follows:-

(i) Subtract from T_L of each immediately following event the "t_e" leading to that event

(ii) Then, select the smallest number (in case there are more than one following event) thus obtained. This value also represents the longest backward time path from the end event.

The T_L values thus calculated are shown in small circles placed below the events and shown in Fig.2.9. The T_L for the start event of the network would then, indicate the latest time by which the job can be started without causing the end event to slip beyond the predetermined target time.

(4) <u>SLACK</u> is the amount of time an event can be delayed beyond its T_E without affecting T_L of the final event and is calculated as

$T_L - T_E$

28 (i) $T_L - T_E = 0$ (zero), this means exactly enough time for job is provided.

(ii) $T_L - T_E$ = +ve, this means some spare time is available and the job is ahead of schedule. It is possible to divert some men, resources from this area to other areas where needed most.

(iii) $T_L - T_E$ = -ve This means enough time is not allowed for the job and the project will take longer time for completion.

Knowledge of this slack is very essential in keeping up a project schedule to the last of its completion. In the last case, network plan needs to be readjusted, if possible, to reduce the time required to complete the project in time. This may be attempted by compressing the critical path in one or more of the following ways:-

(a) By close examination of the elapsed time "t_e" of the critical path activity

(b) By replanning the network by introducing greater "parallelling or concurrency" of activities on the critical path

(c) By transfer or reallocation of slack resources to the extent possible, from the non-critical activities to activities on the critical path.

(d) By addition of more resources to critical activities.

(e) By reducing the scope of activities and/or changing or lowering performance requirements of certain activities, and

(f) By eliminating as a last resort low priority activities if feasible.

(5) ACTIVITY TIMES are not necessarily the event times but they are related as below:-

Earliest Start (E.S.) time = (T_E) of preceding event

Earliest Finish (E.F.) time = (E_S) + Activity duration (t_0)

Latest Start (L.S.) time = (T_L) of succeeding event $-(t_0)$

Latest Finish (L.F.) time = (T_L) of succeeding event

(6) ACTIVITY SLACK is equal to the difference between the maximum time available to do the activity and the time required for the performance of the activity or activity duration (t_0)

Hence Activity Slack = (L.F. - E.S.) - (t_0)

or

= L.F. - E.F.

or

= L.S. - E.S.

Knowledge of activity time is essential to draw up a complete, detailed time table or schedule and to evaluate and optimise the use of different resources and complete network. For a project, illustrated in Fig.2.8, activity times are summarised below:-

TABLE NO.2.1

	t_0	E.S.	E.F.	L.S.	L.F.	Activity Slack
A (1-2)	5	0	5	0	5	0
B (2-4)	4	5	9	5	9	0
C (2-3)	1	5	6	17	18	12
D (4-5)	9	9	18	9	18	0
E (5-7)	6	18	24	18	24	0
F (5-6)	3	18	21	19	22	1

TABLE NO.8.1 -(Contd).

	t_D	E.S.	E.F.	L.S.	L.F.	Activity Slack
G (5-6)	4	6	10	18	22	12
H (6-7)	2	21	23	22	24	1
J (7-8)	1	24	25	24	25	0

8.3. PERT-CPM TECHNIQUE AS APPLIED TO TENUGHAT PROJECT

Now, it will be seen how this technique can be used for time control of an earth dam project. A network is completed (Fig.8.6) herewith a particular reference to Tenughat Dam Project and then discussed below.

8.3.1. ORIGINAL C.P.M. FOR TENUGHAT PROJECT - Fig.8.6(A)

Tenughat Dam Project is scheduled to be completed in five years time (December 1963 to December 1968), each year having working season from October to June (9 months) and July to September (3 months) as non-working period, due to monsoon in the area. Each working month has been considered to include 25 days only on which work will be done, and remaining 5 days have been left out for 4 Sundays and 1 day as holiday for workcharged staff. 'Network' of events and activities have been completed as explained earlier, after planning once, replanning and effecting paralleling or concurrency of activities both on critical and non-critical paths. Latter would help activities on non-critical path to end earlier, thereby enabling project authorities to divert, slack men and machine resources to activities on critical path (shown in thick line). The critical path, is noted to have taken 1250 working days in 5 working years, and that only after this critical path is successfully over, water supply to

TABLE NO.8.1 -(Contd)-

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C. P. M. SCHEDULE FOR TENUGHAT PROJECT

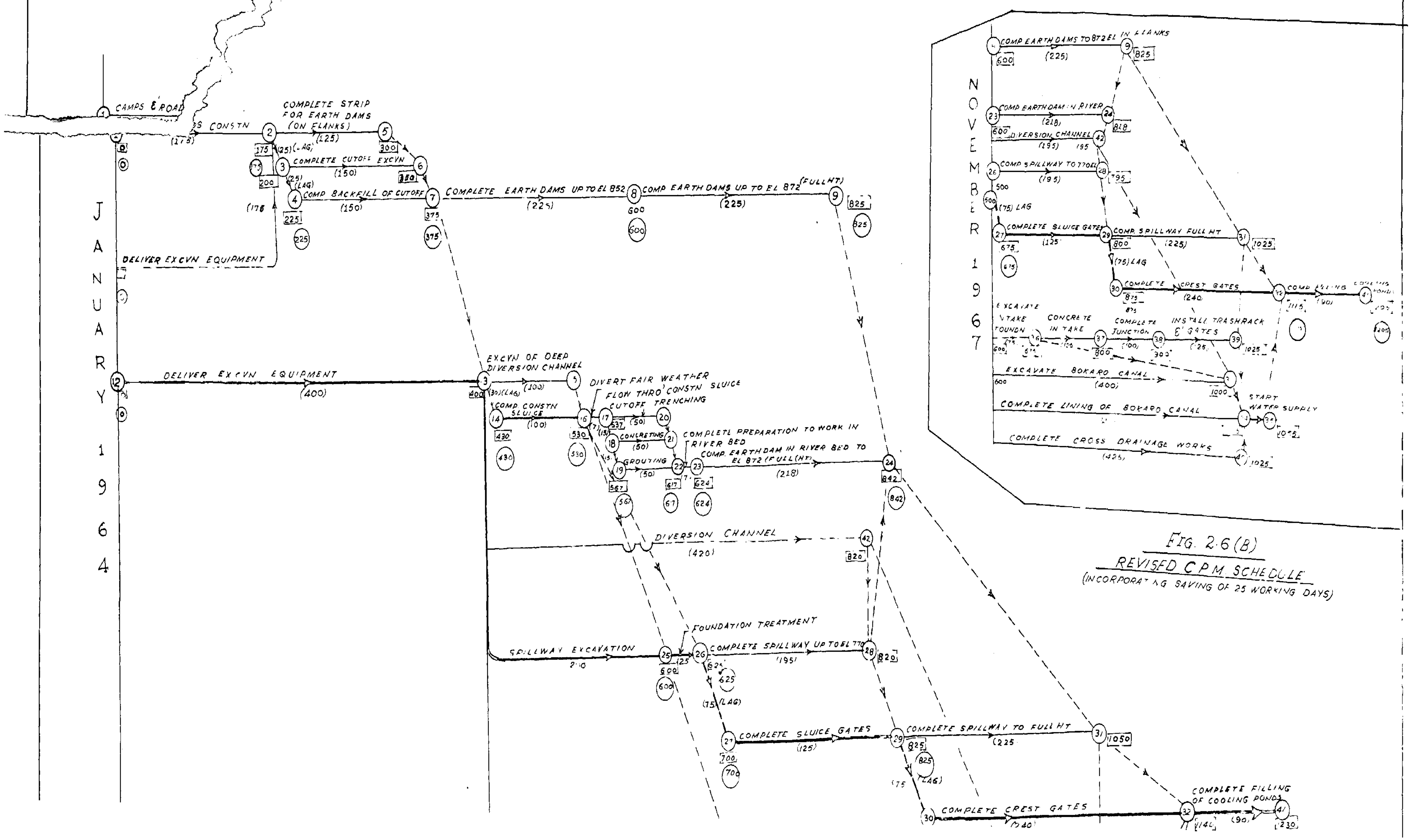


FIG. 2.6 (B)

REVISED C.P.M. SCHEDULE

(INCORPORATING SAVING OF 25 WORKING DAYS)

Bokaro Steel Plant is possible which is so important for the upcoming steel industry there. To control time, completion time of project not to exceed December 1969, this 'network' technique warns project engineers and planners to save time on other non-critical paths, even more than the extent shown in the network in the matter explained below

(1) By further parallelism or concurrency of activities, time of critical path may be shortened; even the elapsed time (t_e) of the critical path activities may have to be examined and reviewed at different stages of work, and if possible total time reduced.

(2) By starting the excavation of deep channel even earlier than 600th day, authorities may afford to be at ease towards the end and still keeping the schedule intact. It appears, procurement and delivery of machines is a handicap to project authorities, naturally if it has to be imported and transhipped from abroad. In the circumstance even old earth moving machines in the country may have to be managed and made in order to start with. Thus time of 600 days earmarked for delivery of goods can be shortened by say, 180 days or so.

(3) Dummy activity (16-23) shows that spillway works (30-37) can start only after both of activities viz. construction sluice construction (16-18) and foundation treatment (26-23) have been completed but River can be diverted for fair weather flow (16-17) and concrete cut off work in river bed (17-20) taken up after construction sluice (16-18) is completed. Now, dummy activity (18-23) can be adjusted to have more time, such that both events 18 and 23 start at 830th working day. (see Network). This will instead a saving of 889 - 830 = 59 days, provided steps as suggested in para (2) are taken. Alternatively, some of the machines from activity (13-15-18) could be diverted to spillway excavation (13-25) after 820th day or even earlier so that total time of critical path viz. 1280 days is reduced by some days.

(4) Slack period both on critical and non-critical paths have been adjusted by showing time lag, in different parts of the job between the beginning of the first and each of the following activities to prevent overlapping at any point in the structure(8) e.g. activities (2-5), (3-6) and (4-6) etc. This is an example showing a "ladder" in which dummy activities or restraints are introduced between the starts of the various activities. These dummies have suitable time allotted to them to prevent overlapping of events. If at a later stage, as the construction progresses, it is found that these provided time lags, can be reduced without upsetting sequence, some slacks reappear. These can be transferred or reallocated to the extent possible, from the non-critical activities to activities on the critical path. This is the other way of compressing the critical path and thus helping time control.

(8) Last but not the least, other ways and means, as explained in para 2.1.8, may as well be optionally tried to achieve time control over the project.

Above network applied to Tenughat Dam Project, gives broad picture of various important works being kept in sequence inter-related with individual and final completion time. As a next step, such individual works, viz. spillway works, earth dams construction, river diversion works (including construction sluice), intake works, Left Bokaro Channel etc. can be broken up into intrinsic activities and sub-networks prepared and examined to see whether any further reduction in time is possible or not. Preparation of such subsidiary networks further helps in exercising better control over time-schedule, in view of broader details of the works being kept ready at a lance. To illustrate, one such case of further breakup will be taken up below vis-a-vis Construction of Construction sluices.

2.3.2. C.P.M. SCHEDULE FOR CONSTRUCTION SLUICE - (Fig.2.7)

In above network, it will be seen that the sum of 't_e' activity elapsed time through path (0-1-3-5-6-7) is larger than that on the other path, (0-1-2-4-5-6-8) and hence the former path determines the total completion time of the job - construction sluice. It will be further seen from the network or the table following, that the event(4) of Excavation of Cutoff has a slack of 31 days but besides that all other events have 'zero' slack. This evidently points to the fact that no further shortening of critical path (i.e. reduction in completion time) is possible, except transferring all or part resources from activity (2-4) to (3-5) and (3-5) for 31 days slack period. Since the work is so important for diverting the fair weather flow through construction sluice and taking up works in river bed, it may be examined in broader details, whether employing more resources at site will be of any avail, seeing limited area at site for the work.

TABLE NO.2.2

Event	Description	Quantity	Period	Activity Elapsed time t_e in working days	Earliest expected Event time	Late-st allowable Event Time	Slack $T_L - T_E$
1	2	3	4	5	6	7	8
1.	Coffer Bank Construct-ion	600 Cft	1.12.66 to 3.12.66	3	3	3	0
2.	Dewatering	One time	4.12.66	1	4	4	0
3.	Wedging Bearing of achieved foundation	550 Cft	5.12.66 to 22.12.66	18	22	22	0
4.	Excavation for cut off trench	6000 "	5.12.66 to 10.12.66	13	10	41	31
5.(a)	Wedging Bearing in cutoff	-	10.12.66 to 21.12.66	0	53	53	0
(b)	Drilling Grouting	1710 Rft	21.12.66 to 21.1.67	31	53	53	0
6.	Anchor Rods & Concret-ing	60550 Cft (Conc)	21.1.67 to 20.3.67	40	93	93	0
7.	Drainage holes	60 Rft	20.3.67 to	7	100	100	0

2.3.3. <u>REVISED C.P.M. SCHEDULE FOR TENUGHAT PROJECT</u> (Fig.2.6(B))

Original C.P.M. schedule for Tenughat Project, Fig.2.6(A) stands for the full project and has been prepared to utilise services of the available project resources in the best possible manner. For example, main aim is to supply water to Bokaro Steel Plant, after completing Earth Dams, Spillway Works and Lined Bokaro Canal. Completion of the three works, should be synchronised such as to enable water supply as early as possible. Though the spillway works lie on the longest path of C.P.M., earth dams have always to be completed higher than the spillway to avoid overtopping and failure of dam by flood water. Naturally, therefore, first preference have to be given in utilising resources to Earth Dams. After they are completed on [illegible]th and [illegible]th days (vide events 9 and 24) machines and personnel have been diverted to spillway works (vide event 31). Similarly, excavation equipments employed on stripping and cutoff parts of the earth dams have been diverted to the deep diversion and construction sluice works on 370th day (7th event) after they were no longer required there. This was also necessary because, unless all additional resources are tapped to complete River Diversion Works early, earth dam in river bed cannot be taken up in time. After excavation of deep diversion channel, machines are proposed to re-employ over spillway excavation. Diversion channel excavation has been planned with independent machinery. Since this involves huge excavation about 3 crore cft and concerted efforts are required to get this completed by [illegible]th days (event 20) by then, spillway gets completed to El.770 and stands ready for diverting monsoon floods over it. The original channel of River is already closed by this time by earth dams constructed in River bed to El.873 Ft. Bokaro Canal excavations are also proposed to be taken up independently with [illegible] sets of machines, having arrived by 400th day (13th event). However, some machines getting free from spillway excavation after 609th day (event 28) can be as well diverted to this job. It is noted that, earth dams already completed to full height (El.872) and spillway completed to (El.770) partial water supply can be met on 1030th day (event 34) after Bokaro Canal including all structures and intake structure are complete by that time. To stick to this target, dummy activities (33-38) can be best availed of. This partial water supply would have been possible, on 820th day (event 28) after spillway had reached El.770, but for the unavoidable delay in execution of Bokaro Canal and Intake Structure, this has lost all chances to be made good. Only now, efforts are to be made to complete the Bokaro Canal etc. by fixed date (1030th day) at least. After this task having been achieved, much of the same personnel and machines will have to be finally diverted to others project or disbanded, since crest gate erection and filling of cooling ponds are the only two left over items, which will avail of only meagre resources at command.

In Fig.2.6(B), a revised C.P.M. schedule has been prepared leaving the items already completed and remaining items to start with November 1967. The steps taken in the above para to divert men and machines in the manner suggested, have resulted in a saving of 25 days (approx) at the start of November 1967 for events 33, 23, 30, 33, 36 and 40. Naturally, the end event 41 on critical path will be completed 29 days earlier on 1200th day instead of 1229th day (of original C.P.M. Schedule) provided nothing unexpected happens. After, affecting 25 days savings, and revising the C.P.M. Schedule (Fig.6.2 (B), some new dummy activities have been brought into play in the manner shown in the Figure. For example, Earth Dam works in River Bed now likely to be over on 818th day (event 20) may well (slack afford some men and machines/resources) for the earth dams in flanks (event 9). Machines employed on Diversion Channel Excavation can be diverted to D[illegible] Canal Excavation after 709th day (event 48) when the River is proposed to be diverted over partly completed spillway upto El.770 etc.

Needless to say, these advantages explained in above, were not possible with either of Gantt Chart, milestone chart, or flow process chart etc. explained earlier.

2.4. CONCLUSION

Of the various methods of planning and control techniques still in vogue, it is considered now imperative to switch over to critical path method which, in essence, is a network to visualise inter-relations between different activities and events of project, with start and finish days and slack hours and 'dummy activities' known at a glance, besides knowledge of activities and events on the critical path, latter having in it, a guide for project engineers to watch it over and complete the project in time or even much ahead of schedule by adjustment and readjustments of the network. We have discussed earlier, the determination of critical path by 'visual method'[1,2] which is easier upto say 60 events. If, however, the network is large, it is advisable and easier to use "Matrix Method"[3] for determination of critical path. "Matrix method" can be conveniently used upto 300 events, though it has, so far, been used upto 700 events. Beyond 300 events, "Computers"[4] were generally used to determine critical path. Further elucidation on different methods of determining a critical path, is not covered by the scope of this Chapter, however they can be referred, in text Books and references given above.

It has been said earlier that this C.P.M. is not very useful for scheduling production control jobs where the processes involved are of a repetitive or routine nature. In one way it can be said to be a failure of C.P.M. on repetitive jobs. But so far our purposes - planning for a project - are concerned, it works alright, whether for the entire project or a part of it.

CHAPTER - 3

FIELD CONTROL

3.1. GENERAL

Before discussing how actual field control can be achieved at site, this is necessary to know what the field control is. Field control governs actual construction operations, and restricts the work to plans and specifications, supplemented by personal mature judgement, suited to a particular site.

It requires correct knowledge of construction materials and for that matter site investigations are to be carried out from time to time. The two aspects of field control - investigations and actual execution - have to go side by side in the field and are broadly covered by the following activities:-

(i) Laboratory investigation of soil compaction
(ii) Field investigation of the soil compaction
(iii) Study of compaction methods
(iv) Placement moisture content determination
(v) Framing construction specifications for compaction
(vi) Field placement control
(vii) Study of pore water pressure in impervious fills
(viii) Investigation of pore pressure in pervious fills due to seismic action.

3.1.1. LABORATORY INVESTIGATIONS OF SOIL COMPACTION

Most standardised method to determine compaction characteristics of a soil in the laboratory is to determine Proctor's Compaction Curve[23]. This curve shows variation of dry density of soil with a variation in moisture content, and gives an optimum moisture content for which dry density for the soil would attain a maximum value. However, this relation between dry density and moisture content is also a function of compactive effort and for that reason, compactive effort is limited in the laboratory by means of standard No. of strikes (say 25). Beyond O.M.C. maximum dry density reduces and the dry density curve approaches 100 % saturation curves at increasing moisture.

A genuine objection to Proctor's test for comparison with field performance is that density moisture content relations do not simulate the kneading effect of sheep foot rollers used for actual compaction of soils. Hence, two alternatives have since been suggested first to incorporate kneading effect in the laboratory and develop kneading compaction apparatuses and the second to prefer moisture density relation by odeometer tests over compaction test. Above kneading apparatuses have been developed by Wilson[81] and Osterberg[82] and others and compactive tests show that the laboratory kneading compaction yields a higher degree of saturation and the results come closer to densities obtained with sheep foot rollers in the field. However, manufacture of such apparatuses on commercial basis is yet to start.

Oedometer tests are employed to investigate the relation between surcharge and dry density or moisture content of soil fill. The moisture content and dry density is selected on the compression diagram corresponding to the surcharge of the respective layer in the dam. This method gives less dry density than the current compaction tests, and hence comparatively less compaction efforts are required for the dam compaction. An important assumption suggested by Mysliwec[83] and involved in this method is that a soil compacted in the fill to a dry density corresponding to the void ratio of this soil obtained in a confined compression test will not suffer further settlement caused by consolidation under increasing construction surcharge. Though the Oedometer tests have been used earlier for Rosshaupten Dam in Germany, fact remains that the standard proctor's test being simple and very common so far is still very helpful in selecting the placement specifications for the fill.

Secondly, since compaction and development of pore pressure are inter-related, laboratory investigations provide a tool for construction pore pressure predictions, so useful for field works. Of late, pore pressure coefficients have been introduced by Skempton[64] and its use for estimating pore pressure in dams very clearly explained by Bishop[65] and Hilf[66]. After having investigated pore pressures and resulting shear strength of soil fill, one can have free hand in deciding placement conditions. For example, construction speed can be increased even by providing effective drainage arrangements even though laboratory investigations show higher construction pore pressures.

3.1.3. FIELD INVESTIGATION OF SOIL COMPACTION

Laboratory investigations reveal general compaction characteristic of a particular soil. Application of these laboratory results to actual field conditions is based on general rules derived from the experience of a large number of dams, built earlier. Their practical applications require specifying type and weight of the roller, layer thickness, optimum moisture and dry density etc. Sometimes other soil characteristics like shear strength and its change with time and permeability consolidation are required to be tested at field on full scale. This gives full confidence while working with prototype. Above test sections of embankments are also useful in studying behaviour of ordinary or special equipments. Apart from that, problems like treatment of the material for drying, moistening, crumbling of clods, excavation methods, number of passes etc. can be solved only by means of the full scale tests in the field. These informations are essentially required during constructions and field investigations on above lines prior to construction make the task easier for field engineer.

3.1.3. STUDY OF COMPACTION METHODS

Construction materials for use on earth dams have quite varied properties and hence, they require different compactive efforts and methods for most efficient working and economical results. Past experiences of working with different equipments and construction materials have given some generalised and useful informations regarding compaction methods for the construction engineers. Still there is scope for developing new and better equipments for soil compaction apart from trying old ones on test sections and comparing relative results to decide their efficiency, number of passes, weight of rollers and optimum moisture content for best results. Different compaction methods are dealt with in detail at Chapter - 4.

3.1.4. PLACEMENT MOISTURE CONTENT DETERMINATION

Deciding placement moisture content for soil fill is governed by so many factors viz. deformability of the foundation, characteristics of the materials, climatic conditions, time schedule of works etc. Though laboratory investigations may give an optimum moisture content at which maximum dry density of soil is anticipated but to achieve that very density at field for all layers, of fill may be very difficult for practical reasons. Hence a tolerance of O.M.C. is given for site works. Again, this tolerance or limits of O.M.C. depends on the factors enumerated as above.

Selecting placement moisture much below on dry side is confronted with dangers of the dam having excessive settlement on saturation while there is a limit on the wet side, beyond which operation of equipments becomes impracticable. Between these two limits placement moisture is generally chosen, with due regards to other factors influencing the purpose, stability and economy of the dam. For example, in the dry region where water may be scanty it is not possible to place the fill wet, while in the wet climate it will be impossible to dry the fill material. Wet construction though involves risk of construction pore pressures, being developed, it also helps working with sheep foot rollers upto a earlier limit. Nevertheless either of the two - dry or wet construction - can be chosen, suited to field conditions, in deciding placement moisture content.

In Japan, due to adverse climatic conditions prevailing there, the fills are placed with their natural moisture content mostly wet of optimum and the implications of this conditions are considered in design. The construction rates are slow and light tamping rollers (260 p.s.i.) are used.

In Ceylon, also due to adverse wet and cool climate of the country, wet construction is the only alternative.

In Serre Poncon Dam (France), wet construction (3 % wet of optimum) had been chosen, the maximum moisture having been limited by the practicability of equipment operation.

3.1.0. [illegible] CONSTRUCTION SPECIFICATIONS FOR COMPACTION

For compaction control two specifications are possible

(i) 'Performance' type, and

(ii) 'Quality' type

(i) Performance type specification is one in which the type of equipment, the number of passes, the moisture content and the thickness of the layers are stated with provision for payment of extra roller time to comply with variation of soil characteristics.

(ii) Quality type specification is one in which the final properties of the compacted fill are prescribed.

Both the types of specifications when correctly adhered, will give similar results. However, performance type specifications give a clear picture of actual construction works and is based on comprehensive study of the behaviour of the soil characteristics and of various types of equipments to be employed on the field. The quality type specifications leave to the choice of the constructors to decide on the employments of compaction machinery etc. specifying only the acceptable density and moisture content values.

3.1.6. FIELD PLACEMENT CONTROL

Field placement control of rolled fill is primarily meant to ensure that the density and moisture content are obtained with the lowest scattering from the values specified in the design resulting in a homogeneous fill. The sooner the density and moisture content of the rolled fill are determined, the better. Different methods of such determination are discussed ahead . Besides laboratory tests visual observations of behaviour of the rolling process, inspection of borrow area by comparing samples, help much in controlling field placement. In controlling field placement, frequency of sampling and limits of compaction factors are decided based on size of the project and available hands. This is also discussed ahead.

3.1.7. STUDY OF PORE WATER PRESSURE IN IMPERVIOUS FILL

This is dealt with separately in Chapter 6, Article 6.2. Stand pipes, piezometers tips connected to bounden gauges, electro acoustic manometers are some of the types of pore water pressure installation that can be used for measuring construction and post construction pore pressures. Especially in rapid construction works, use of this installation becomes inevitable. Such installations in existing dams show that the pore pressure may present a serious problem affecting stability.

3.1.8. STUDY OF PORE WATER PRESSURE IN PERVIOUS FILLS DUE TO SEISMIC ACTION

This is dealt with separately in Chapter 6, Article 6.1. Pore pressures are not normally expected in gravelly and sandy pervious fills. However, due to seismic force, during earthquakes, the porosity of the fill may cause sudden decrease in porosity causing temporarily increased pore pressure. This may subsequently affect stability of slopes of such pervious fills. Especially in case of fine sand, there are chances of absolute collapse of the structures, known as liquefaction. Now a days, such likelihood can be ascertained by simulating seismic conditions at site of desired magnitude and results noted for liquefaction. In case of any indication of the pervious fill being settled excessively under seismic loading, the fill can be compacted by blasting, vibroflotation[61], compressed air injection[72] or electro-osmosis[70] methods whichever is economical.

After having studied above problems related to investigations and execution of high earth dams, actual field control can be achieved by having two separate organisations - quality control organisation and construction organisation. These are dealt below-

3.2. QUALITY CONTROL ORGANISATION AND ALLIED ITEMS

Following are some of the important functions of quality control organisation:-

3.8.1. FUNCTIONS

(i) To study design assumptions and specifications of earth dams and to ensure them in construction by performing various tests. If there is any deviation from the design assumptions affecting stability of structure, it can be either stopped or suitable corrective measures taken in time in the design, and specifications.

(ii) To keep systematic records of actual characteristics of soils in different layers of earth dam for post construction investigations, if any

(iii) To instal instruments (e.g. Cross Arm, Base Plate, Horizontal Movement Indicator, Surface Movement Indicator, Piezometer Hydrostatics and Earth Pressure Cells) for study of structural behaviour of dam during and after construction.

(iv) To suggest depth of cutoff trench based on field permeability test and to suggest foundation treatment.

(v) To investigate special problems of foundation (if any) which was not anticipated during project investigation prior to design and construction.

(vi) To ensure that density and moisture are obtained with the lowest possible scattering from the value specified in the design resulting in a homogeneous fill in any zone of earth dam. Moisture distribution in each zone should be as uniform as possible in each shift. Materials used should conform to requirements.

(vii) To ensure proper bonding of embankment with foundations abutments between layers and against structures adjacent to the fill.

3.8.2. ENFORCING QUALITY CONTROL

For efficient discharge of the different functions as above elucidation on following points will be necessary. After understanding the problem, quality control can be enforced at site by tech. personnel.

(i) The Duties of the Quality Control Inspectors

(ii) Proforma, Reports and Records

(iii) Plans, specifications for compaction and sampling frequency

(iv) Instrumentation in Dam

(v) Field investigations for foundation treatment

(2) DUTIES OF QUALITY CONTROL INSPECTORS

Number of personnel in quality control organisation is limited by the quantum of work. In India, strength of this organisation is known to vary from State to State and even from Project to Project(8,11 to 19). However, for an earth dam work, quality control inspectors will have to be allocated against the following works:-

(i) Borrow Area Control - Variables are water content, density and chemical composition of soil

(ii) Embankment Control - Variables are placement moisture content and uniformity of moisture in soil, condition of surface of layer after rolling and before receiving earth

(iii) Compaction control - Variables are roller characteristics (weight, type of tampers, contact pressure, air pressure in case of pneumatic roller etc) number of passes, thickness of layers, maximum size and quantity of rock in the material etc.

(iv) Field Laboratory works

Duties of individual inspectors are defined below:-

BORROW AREA INSPECTOR

Sees that soils are taken from correct borrow areas and from specified depth only. He also determines in-situ density and moisture content of borrow area soil in Laboratory and compares it with specified values.

EMBANKMENT INSPECTOR

Directs placing of soil at correct places and in horizontal layers of the specified thickness, conforming to grade and alignment, and assures that the soil has been mixed with required amount of water to facilitate compaction.

COMPACTION INSPECTOR

Assures that required compaction either by performance type or quality type specifications are enforced at site.

LABORATORY ASSISTANTS

Takes samples of compacted fill at regular interval and checks it in the laboratory whether correct compaction has been obtained. If not, he also suggests corrective measures. Besides, he also checks material of Borrow Area from time to time and compares it with original properties of soil, prescribed for use on the Dam.

(ii) PROFORMA, REPORTS AND RECORDS

Proforma are required to keep systematic records of the progress and quality achieved during earth dam construction in different layers. Items of proforma may also vary from project to project as per requirement and type of construction viz. Rolled fill, Hydraulic fill or Dumped and Sluiced Rockfill. For Rolled fill construction, following items may form amongst others part of the quality control proforma as decided by Project authorities.

QUALITY CONTROL PROFORMA

I - PLACEMENT OF FILL

(i) Location of fill and Elevation

(ii) Thickness of fill layers

(iii) Number of Roller Passes

(iv) Placement moisture content

(v) Dry density of compacted fill

(vi) Compaction efficiency

II - BORROW AREA EXCAVATION

(i) Borrow Area Location

(ii) Type of Excavation

(iii) Depth of Cut

(iv) % Moisture Content

(v) Dry density of borrow area material

PROGRESS PROFORMA

This proforma shows progress of Earth Dam construction in terms of

(i) Daily progress

(ii) Upto-date progress

(iii) Upto-date progress with reference to total quantity involved

(iv) Short fall or excess in progress over daily targets

(v) Shortfall or excess in progress over cumulative targets on the date of reporting

(vi) Reasons of shortfall or in excess progress

Above proforma, when filled in correctly, becomes record of the work done, prior to date of reporting. Such reports may sometimes be accompanied by short remarks if some point remains uncovered by above items.

Hirakud Project Authorities had maintained quite a systematic record of the daily reports of the tests carried out for Earth Dam both in Laboratory and in the Field, and these have been kept at Hirakud Research Station even now. Such records become a good guide for Maintenance Engineer later on as he can understand the properties of soil as could be achieved on the dam vis-a-vis properties prescribed in designs and their effect of variation on actual behaviour of Dams.

(iii) PLANS SPECIFICATIONS FOR COMPACTION FACTOR AND SAMPLING FREQUENCY

(a) PLANS

Plans and drawings are usually prepared in Designs Directorate. There might be some omission or commission and those should be immediately reported back to Directorate and get corrected. Once the plans and drawings are finalised, these should be strictly adhered to at site.

(b) SPECIFICATIONS FOR COMPACTION

Laboratory prescribed values of compaction may not be economically feasible at field and hence field compaction factor needs to be determined by striking a balance between resources at command and the design needs. Generally 98 % compaction at 2 % deviation, either on wet or dry side of O.M.C. of impervious fill and 70 % compaction for semipervious fill (in profusely saturated state) are adopted for passing a fill layer over embankment. It is understood that for impervious fill compaction, though O.M.C. is a controlling factor, the compaction can still be improved by controlling compaction effort (discussed in Chapter - 4). Hence, by reducing O.M.C. even by 2 % and increasing compaction effort, the same compaction can be obtained which could have been possible from compaction of soil fill at O.M.C. with more number of passes (an uneconomical proposition) by a standard roller. Following are some of the practices followed for dams.

	Compaction Ratio	Deviation from O.M.C	Remarks
(1) <u>U.S.B.R. Dams</u>			
(a) 0 - 150'	98 %	0 to 2 %(-)	
(b) 150' - 300'	96 %	1 to 2.5 %(-)	
(c) 300' - 450'	94 %	1.5 to 3 %(-)	
(2) <u>INDIAN DAMS</u>			
(a) <u>Nagarjunasagar</u>			
(i) Left earth dam (85' high)	98 %	0 to 1 %(+)	Machine Compaction
(ii) Buggavugu Dam (98' high)	98 %	0 to 2 %(-)	"
(b) Tenughat Dam (142' high)	98 %	0 to 1 %(±)	"
(c) Hirakud Dam (195' high (i)	98 %	0 to 2 %(±)	"
(ii)	95 %	0 to 1 %(+)	Manual Compaction

It would appear from above that Indian Practice has now standardised to obtain 98 % compaction at 2 % deviation either on wet or dry side of O.M.C. But, it would be better to prescribe compaction in terms of various classifications of soil, instead of marking a dead line of 98 % compaction for all kinds of soil. India is a big country, and quite a variation in soil properties and types is to be noted from one region to other. In that context a more rational compaction can be as given below. Though it is suggested by Sowers(6) in context of U.S. Soils, it will hold good for Indian Soils too since it is based on unified classification of soil, which India has also accepted as a more rational approach to soil classification. The classification is valid for High Earth Dams more than 100' Height.

<u>For Earth Dams over 100' Ht</u>

Unified Classification of Soil	Compaction Ratio	Moisture Control Deviation
(i) CL & ML	100 %	1 to 2 % (-)
(ii) SC	90 %	1 to 2 % (-)
(iii) SM, SP, GC, GM	98 %	1 to 2 % (-)
(iv) GW, GP, SW	97 %	1 to 2 % (-)

However, in case of (iv) GW, GP, SW, Indian Practice has so long to adopt U.S.B.R. or U.S. Army Corps Practices. Sherard(7) also favours use of any of the two U.S. practices for Relative Density or Density Practice as discussed below:-

(i) U.S.B.R. Practice, Relative Density (R.D.) = 70 %

where
$$R.D. = \frac{e_{max} - e_o}{e_{max} - e_{min}} \times 100$$
$$= \frac{D_{max}\,(D - D_{min})}{D\,(D_{max} - D_{min})} \times 100$$

(ii) U.S. Army Corps Practice - Density Ratio (D.R.) = 60-80 %

where
$$D.R. = \frac{D - D_{min}}{D_{max} - D_{min}} \times 100$$

In either case, the compaction can be easily achieved by saturating sand or gravel profusely. It has been noted that changing dry density of sand from 80 % to 90 % of maximum density, brings about a change from 60 % to 80 % in relative density. Obviously, relative density or density ratio being more sensitive to density changes is a better tool to control compaction of pervious soil (GW, GP, SW) at field than the density compaction ratio. In this context Severs Classification, as above, may be adopted as such for other soils while that for (iv) U.S.B.R. or U.S. Army Corps Practice be better followed.

(c) SAMPLING FREQUENCY

To pass a fill layer, there must be some yardstick as to what will be frequency of sampling to O.K. quality of the work done. Though no fixed criteria can suit one and all cases of fill compaction, and inspector's experience and judgement will count specially in doubtful cases, project authorities will have to decide upon it based on the volume of the work to be done and availability of staff, in the project. A comparison of sampling frequency of 195' high, Hirakud Dam (India) with that of U.S. Bureau of Reclamation Dams may be of interest here(8)

Purpose of Sample	Frequency		Remarks
	Hirakud Dam (India)	U.S.B.R. Dams	
1. For Laboratory Records	-	One in 30000 cu.yd	
2. For passing fill layers	(a) One in 3700 cu.yd. (10,000 Cft) (N.B. At Nagarjuna Sagar one in 5000 Cft)	One in 2000 cu.yd or One test per shift of work	In case of undoubted compaction viz. over embankment
	(b) 50' - 100' apart	Inspector's decision	In cases of doubtful compaction viz in filling cutoff trench abutment and junction of

It appears, U.S.B.R. Dams have been maintaining laboratory records besides day to day records to pass fill layers. No doubt, the more the records, and more the frequency of sampling the better. But for India, having limited resources, Hirakud standard of sampling one in 10,000 cft is considered adequate and may be followed for other Indian High Dams too. However, as in case of Hirakud Dams, it may not be practicable to maintain all daily records for decades to come after construction of dam big or small, but it will be desirable to retain at least results of one representative test of one fill layer for future guidance.

(iv) INSTRUMENTATION IN DAMS

Instrumentation is an important tool to study the behaviour of dam during and after construction and compare it with design provisions. By measuring from time to time, settlement pore pressure and lateral displacement of Dam, it is possible to improve design criteria and assumptions for future dams to be built. In some way, pore pressure readings do help in improving drainage arrangement also during construction period. If the rate of construction is quite rapid and it is felt that dissipation of construction pore pressure is going to take longer time at its own accord - artificial drainage becomes obligatory. Similarly, settlement studies forewarn whether the dam has excessively settled during construction period and whether it warrants measures to be taken to improve compaction of the embankment or foundation.

(v) FIELD INVESTIGATIONS FOR FOUNDATION TREATMENT

Such investigations include digging bore holes and obtain drill data to have intimate knowledge of underground geology on which dam is to be built, followed by field permeability tests, to decide depth of cut off trench and also foundation treatment. Field permeability tests are also helpful in confirming definite ratio of coefficients of permeability in different zones of a zoned fill embankment. There are different methods available to determine in-situ permeability tests out of which U.S.B.R. methods - Designation E-18(0) for field permeability tests in bore holes - is most commonly adopted in India for field exploration purposes. Designation E-19 of well permeameter is useful for day to day work in the embankment. An extract of some of the Swedish methods, used in Gosshonemalp Dam for passing area for further dumping is given at Appendix 5.1.

5.3. CONSTRUCTION CONTROL ORGANISATION AND ALLIED WORKS

This Organisation is responsible for construction works and looks after following, in general:-

(i) Foundation Preparation and General Treatment

(ii) Surface Treatment of Rock - Foundation and Abutments

(iii) Construction Operation

(iv) Construction Progress

GENERAL

8.3.1. FOUNDATION PREPARATION AND TREATMENT

This is one of the important responsibility of the Construction Engineer. His duty lies in ascertaining the actual foundation condition while construction is taken up and if required, necessary changes get incorporated in the earlier designs, which are always based on a limited investigations report, and hence may not always be giving correct state of affairs, underground. His next responsibility is to ensure that adequate bond between foundation and the earth dams are made, and enough safeguard taken against foundation failure, stripping of area, providing cutoff by clay backfill or concrete diaphragms or sheet piles or grouting and compacting pervious foundation by whatsoever means, if the dam is susceptible to excessive settlement or liquefaction. While stripping ensures better contact and compaction of earthfill over foundation, this should not be very orthodoxically taken in context of modern high dams, because so many kinds of roller and vibrators are available there in the market that achieving desired compaction is no more a problem now. Stripping, however, removes roots and woods in the foundation. Stripping is considered satisfactory, if roots or woods not yet removed from below the embankment does not exceed 10 %[78]. Incidentally on earth dams this quantity may be allowed upto 5 %, the lesser, the better. Justin[70] expressed the opinion that as much as 8 % also is permissible sometimes and also small bags of vegetable matter may increase the impermeability of soil.

Other foundation treatments, as envisaged above have to be taken care of in context of conditions obtaining in a particular project only, but if a positive cutoff is adopted it should be ensured that the cutoff is taken at least 5' inside the impervious stratum and if considered necessary impervious stratum (soil) or rock below this elevation may be grouted to reduce seepage and increase stability of structure by increasing seepage path. Alluvial grouting has been recently practised successfully at Serre Poncon Dam (France)

(87)

and High Aswan Dam (Egypt). Judging from their performance Indian Dams can as well adopt it. Hota Dam finally took recourse to alluvial grouting there after piles had failed earlier in the pervious foundation consisting of sand, boulder and clay, extending to 60' (approximately). Concrete Diaphragm by Rodio Marconi Method is considered useful for such pervious foundation but experience at Gandak Barrage (Bihar) has shown that unless the pervious foundation comprises of sand only, it is not very desirable. It does not give a good performance in boulders and shingles foundation strata. In case of cut off trench being back filled by impervious clay, care should be taken to provide enough bond between the fill and sides of the trench, which is often missed during construction. Similarly, laying of impervious fill under subsoil level should be started only after the trench is sufficiently dry. Sumps or wells employed for dewatering should be away from the cut off area. Another necessary precaution also with impervious layer in foundation overlying pervious stratum. Such impervious layers are likely to give way and get punctured after construction under greater uplift. If the top impervious layer is relatively thick say 6'-8' it can be economically excavated and backfilled with pervious material in the pervious zone of earth dam but no extra treatment is required under impervious zone of the dam, where cutoff is already taken to an impervious stratum. If the impervious layers cannot be taken out economically, relief wells may be provided later, on seeing the performance of the Dam. Sarda Sagar Dam in U.P. is one of the present examples in India, where the Dam founded on stratified foundation was not giving satisfactory service. Relief wells provided after construction have been learnt to be satisfactorily working now.

It is worth mentioning in this context that Nanak Sagar Dam (U.P.) failure in the first week of September 1967 resulted mainly because no adequate treatment was provided for stratified foundation beneath the dam. It is learnt the top 3 m depth comprised of impervious stratum overlying 9 m poorly graded sand. Treatment for foundation in this case lay in removing top impervious foundation below the shell material of the dam and filling with the same material as that of the pervious shell and adequate provisions of filters D/S or providing adequate relief wells, after construction. In case of Panshet Dam (Maharashtra), insufficient compaction at outlets and abutments contributed to its failure, even during its construction. Ahraura Dam (U.P.) failure also resulted from the trouble of seepage developed at junction with masonry sluice. Treatment required at junctions and abutments of High Earth Dams are discussed below.

(ii) SURFACE TREATMENTS OF ROCK FOUNDATIONS AND ABUTMENTS

After underground treatment of foundations against excessive seepage, discussed above, the rock surface against which the impervious core of the dam is to be compacted and sealed, must be treated. If rock is soft under the dam core, treatment of the abutments and foundation consists simply of excavating a smooth surface against which the impervious soil of the embankment core can be compacted with heavy rollers. If such soft rock is likely to deteriorate when exposed to weather such deteriorated surface should be stripped in small strips just before taking up of backfilling or should be protected by sprayed coats of asphalts.

In case of rockfill dam, there is no danger of such rocks under impervious core to deteriorate when covered. However, for such foundation rocks coming under rockfill are exposed to air circulation and likely to deteriorate during life time of the dam. No satisfactory solution for all sites of this type is available. Flattening of rockfill slopes as much as in case of earth dams may help the situation. It has been however found that a thin coating of asphalt is generally much more satisfactory than gunite or concrete slabs in protecting the soft rock from deteriorating during construction[7]

In case foundation rock is hard, treatment lies in smoothening up the surface sufficiently to ensure better bonds between dam and the foundation by better compaction. Such hard rocks are treated by removing overhangs by light blasting or jack hammers, filling depressions with concrete or hand tamped earth and if rock is fractured cleaning out and filling the surface cracks with cement grout. Rock treatment in general, comprises of following four steps[7].

(i) Cleaning of loose materials over rock surface and shaping the rock to as smooth and regular a surface as practicable.

(ii) Washing exposed surfaces with water and/or air jets

(iii) Sealing the cracks to rock surface

(iv) Compacting first few lifts of earth embankment against prepared rock surface.

CLEANING AND SHAPING

Cleaning and shaping of foundation and abutment rocks are completed first with earth moving equipment as far as possible and finally by manual labour. Sometimes small charge blasting is taken recourse to remove overhangs. Rock bolting may sometimes be also useful with steep abutments to improve stability of the slope during construction.

(ii) WASHING

Hand cleaned surfaces are thoroughly washed with powerful water jets from higher elevation to lower elevation and final cleanup generally given by air jets.

(iii) SEALING THE CRACKS

Such cracks comprise of

(a) Pot holes or depression of limited extent

(b) Widely spread cracks due to joints and fractures

(c) Open cracks.

Pot holes or localised depressions can be filled with hand compacted soil or cement. If pot holes are cracked in sides or bottom, cement grout can be used and the holes can be backfilled with compacted earth. In case pot holes are too many and closely spaced a concrete slab pour over the entire rock surface will be a better proposition.

Widely spread cracks can be treated with cement grout gunite or high water content clay compaction below the earth core.

Surface fracture can be sealed by sand cement slurry grout after clearing the rock surface. Fault zone or large cracks are excavated and backfilled with concrete upto several feet.

(iv) EARTH COMPACTION AGAINST ABUTMENTS AND ROCK FOUNDATIONS

While compacting rock against steep rock abutments the construction surface of the embankment should be sloped away from the rock or walls for a distance of 8 to 12 ft at an inclination of 6:1 or steeper. This helps better compaction by allowing roller to act more directly to compact the earth against the rock surface. Rolling of layers should be done in thin layers with smaller equipments if large rollers cannot be employed because of little space. Sheep foot rollers are not very useful for first few lifts of the construction. The more irregular the rock surface the more desirable it is to compact the soil at a high water content. Relatively fewer material should be used to make the junction impermeable.

5.3.2. CONSTRUCTION OPERATION

Construction operation is more or less administrative concern and its efficiency depends upon right selection and employment of excavation equipments. Construction operation includes watching their proficiency and ensuring the work to specifications. With higher and higher earth dams being taken up for construction, the construction is becoming more and more mechanised. Variety of excavation and construction equipments have come to the market and

and others will continue to come as improvement over previous ones. Construction Engineers will have to keep their mind open to adopt them, if they suit their needs and even ask Construction Firms to manufacture to their prescribed specifications, if the one available in the market is not suited to their conditions. As regards specifications, no rigid rules can help the construction. Specifications should be framed after understanding the availability of equipments in hand, site conditions and above all design considerations.

5.3.3. CONSTRUCTION PROGRESS

Construction progress apart from being dependent on proficiency of excavation and construction equipments, availability of man power resources and deployment of construction materials etc in case of high earth dams, depends upon how early the fill layers can be passed before taking up another layer. This is a great bottleneck in high earth dam construction and secret lies in reducing time to ascertain density and moisture content of the fill layers. Different methods have been evolved, but in India, still no recognition has been given to rapid control methods, and in most of the dams, yet emphasis is laid on determining actual moisture content of fill soils and its dry density.

In subsequent paras, a review of all such control methods is given along with a discussion of their relative merits and demerits. It will help a construction engineer to decide and adopt only a rapid control method, suited to his need and job, instead of still going for rigorous and time consuming method followed earlier.

Accelerating construction progress also depends upon time control of different operations as a whole and this has been recently possible by introducing concept of critical path method (Chapter - 8). This method utilises man and machine resources of the project to the fullest possible use.

Construction progress also depends upon proposed River Diversion works, target of works and natural conditions. But all such factors involve more of planning than of construction. Among them, only rapid control methods will be dealt here in detail.

8.3.4. METHODS FOR DESTRUCTIVE MOISTURE CONTROL.

Moisture content of soil is defined as %age weight of water content per unit weight of dry soil. Initial weight of wet soil can be determined directly on weighing balance but drying of wet soil takes time, before which weight of dried sample cannot be taken. New methods have, therefore, been recently evolved, either to dry the soil sample quickly or to estimate the %age water content without actually drying the soil sample. This estimate should of course be fairly accurate for all practical purposes. Following methods summarised also at Table No. 8.1 have ~~recently~~ been used in different countries but they all have their own limitations. None of the methods are therefore universally accepted. Theory, procedures and merits, demerits of all such methods are briefly described in the following para.

(i) OVEN METHOD

In this method, soil is kept over some pan and heated by an ordinary oven (gas or electric) till its weight becomes constant and no more water particles remain back with soil to be evaporated. The method is simple but takes about 12 hours time to determine dry weight of soil. Now a days, some "Temperature Controlled Ovens" are available in the market, but this also has not been able to reduce time further than 6 - 8 hours.

TABLE No.3.1

CHARACTERISTICS OF DIFFERENT METHODS FOR DETERMINING FIELD MOISTURE CONTENT

	METHODS FOR FIELD MOISTURE DETERMINATION																			
	Designation Oven Use ~~[illegible]~~ Eo of U.S.B.R. Earth Manual	Forced Draft	Open Pan	Alcohol Burning	Proctor Penetration Designation E-22 of U.S.B.R. Earth Manual)	Hilf	Compacted Wet Unit Weight	Nuclear	Distillation	Calcium Carbide Pressure	Calcium Carbide loss in weight	Pressure Pycnometer	Pycnometer	Alcohol Solution	Refractive Index	Electrical Resistivity	Electrical Capacitance	Turnbull Drop Test (Australia)	Temperature rise on addition of Sulphuric Acid	Empirical Method
(1) Time (Minutes)	720	30	45	35	10	60	10	10	45	10	10	10	10	10	10	10	10	10	10	30-60
(2) Usable With																				
(a) Non-cohesive material	Yes	Yes	Yes	Yes	No	No	Partly	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	no
(b) Cohesive material	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
(3) Usable																				
(a) At Sampling Site	No	No	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
(b) Without identifying soil	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes
(c) Without compaction curves	Yes	Yes	Yes	Yes	No	Yes	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
(d) Without calcination curves	Yes	Yes	Yes	Yes	No	No	Yes	Yes	Yes	No	No	Yes	Yes	Yes	No	No	No	No	No	Yes
(4) Use in field [illegible]	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	No

(ii) ELECTRIC FORCED DRAFT HEATER

This type of heater is an improvement over earlier temperature controlled ovens and is able to dry 80 gm soil samples in 30 minutes time only.

(iii) OPEN PAN

The soil is heated in this method on an open pan by ordinary heating arrangements and may take about 40 minutes time.

(iv) ALCOHOL BURNING METHOD

This method consists in mixing denatured (Methyl) Alcohol with wet soil to form a slurry in a perforated metal pan and igniting the alcohol to burn off. Since alcohol burns off completely at 150° - 180°C, temperature of soil does not exceed this. Following steps will have to be followed in this method time required being 10 - 20 minutes only

(1) Weigh perforated pan

(2) Obtain representative sample of soil

(3) Place sample in perforated pan, weigh and record weight.

(4) Place perforated pan in larger pan and stir alcohol into the soil sample with a glass rod until the mixture has the consistency of a thin mud or slurry. Clean rod.

(5) Ignite the alcohol in the outer pan and in the sample and burn off all alcohol.

(6) Repeat the process three times or until successive weighings indicate no reduction in weight, each time burning all alcohol.

(7) Weigh perforated pan and dry soil; record weight after final burning. The weight of dry soil equals this weight minus weight of perforated pan.

(8) Calculate moisture content, thus

$$\%\ \text{Moisture Content} = \frac{\text{Wt of Wet Soil} - \text{Wt of dry soil}}{\text{Wt of dry soil}} \times 100$$

$$= \left(\frac{\text{Wt of wet soil}}{\text{Wt of dry soil}} - 1\right) \times 100$$

(v) PROCTOR PENETRATION RESISTANCE METHOD

Resistance to penetration developed by the compacted soil is utilized in this method to assess the % age moisture content presented in the fill. Steps involved in this method are:-

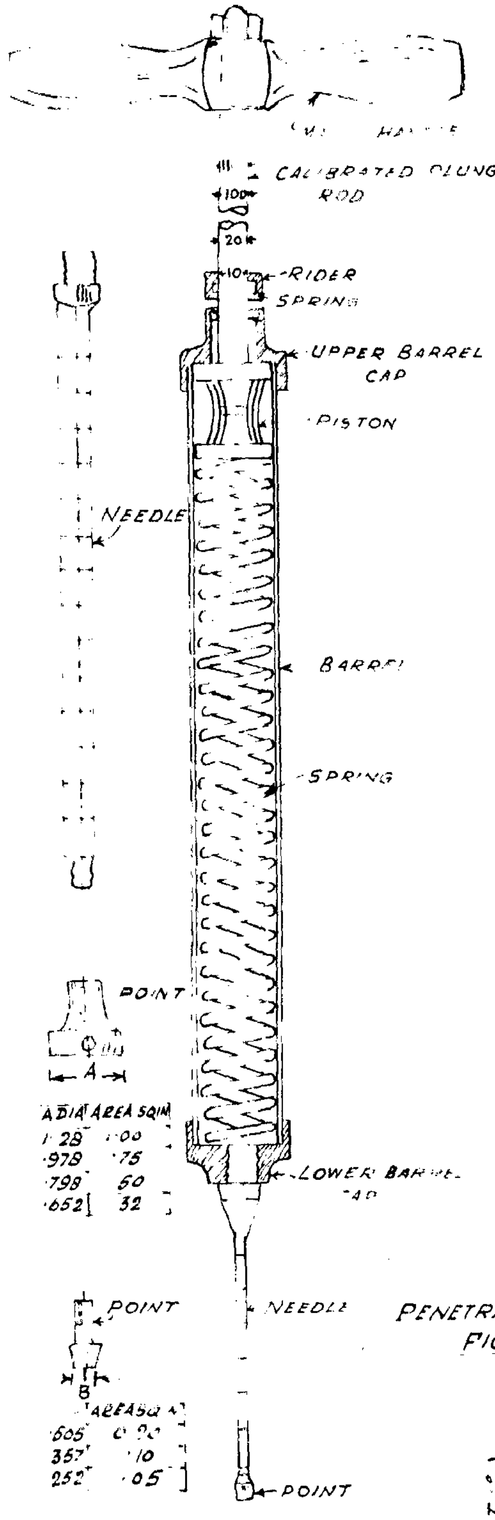
(1) Taking a soil sample from the rolled filled work

(2) Compacting it into a standard compaction mould

(3) Weighing to determine the wet unit weight of the soil

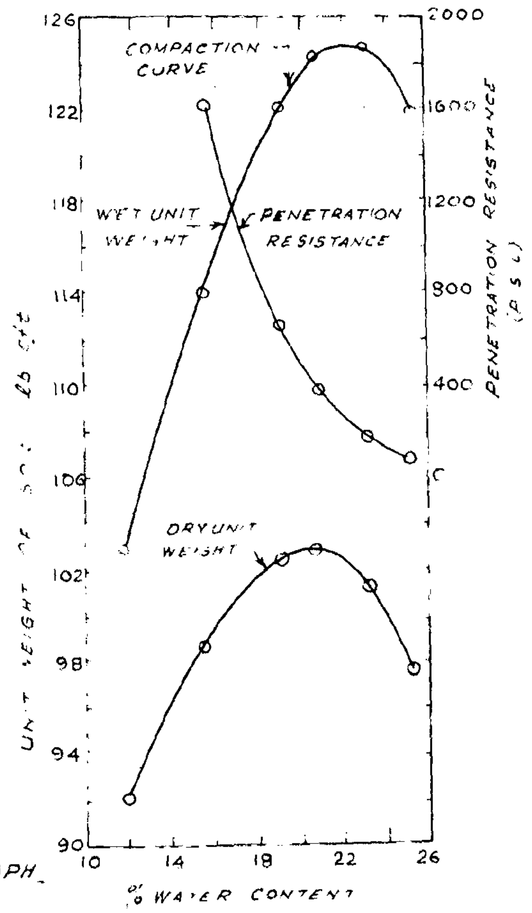
(4) Measuring the penetration resistance of the soil in the mould with the soil penetrometer (Fig.3.1), and

(5) Determining the moisture content from a previously established chart that relates wet unit weight, penetration resistance and moisture content like one shown in Fig.3.2. These steps take 10 - minutes time and the results are sufficiently accurate for most field purposes. The method is however, suitable for fine grained soil, because coarse

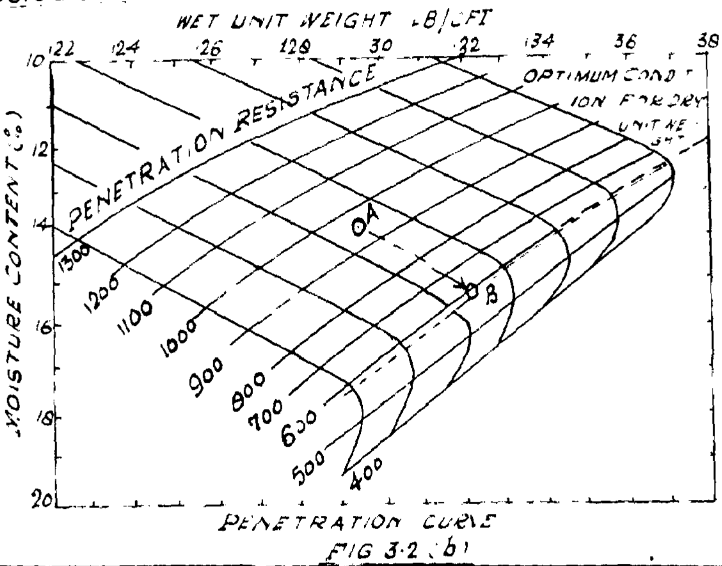


SOIL PENE-TROMETER
(PROCTOR TYPE)

FIG. 3·1



PENETRATION GRAPH
FIG. 3·2 (a)



PENETRATION CURVE
FIG 3·2 (b)

sand and gravel may cause erroneously high resistance readings.

Graphs as in Fig.3.8 is normally prepared from the results of pre-construction compaction and penetration tests. For determining penetration resistance, penetration needle is pressed into the compacted soil of the Proctor's mould at a uniform rate of ½ inch per second to a depth of 2½ in, and the resistance in p.s.i. come on calibrated springs, (Fig.3.2). β moisture content can be read from the graph (Fig.3.8) against this penetration. To make the data more usable in the field for moisture determination and for soil identification, a plot similar to that in(Fig.3.8 b) is generally made. If, for example, the wet unit weight of a sample recompacted in a compaction mould is 120.6 lbs/cft and the penetration resistance is 650 p.s.i. these data can be plotted as point A and the moisture content read directly as 13.0 percent. The chart can also be used to determine the optimum moisture content and maximum unit weight of the soil tested. This is done by sketching in a compaction curve passing through point A and intersecting the line of optimum conditions at point B. The moisture content at point 'B' is optimum maximum dry unit weight is the wet unit weight at point B divided by one plus optimum.

(vi) [illegible]

This method has been discussed ahead along with determination of Dry Unit Weight of soil fill.

(vii) [illegible] METHOD

Steps involved are:-

(1) Obtain a sample from rolled earthwork

(2) Recompact it into a mould

(3) Weigh this mould and soil to determine the wet weight, and

(4) Plot the wet weight on the compaction curve, prepared earlier.

The moisture content can be read off directly, from the Compaction Curve (Fig. 2.8). For example moisture content of a characteristics soil against 120 lbs/cft wet density, is 20.1 %.

(viii) NUCLEAR METHODS

This method is discussed ahead along with determination of wet density of soil fill.

(ix) DISTILLATION METHOD

This method consists in

(1) adding Toluene (or xylene) chemical to the soil sample

(2) heating the mixture to drive off water content of soil along with Toluene, and

(3) condensing the vapour in the collection tube.

The water is heavier of the two liquids, and can be measured directly in the collection tube. The Toluene floats on the water and is recirculated to the still where it prevents the temperature of the mixture from exceeding its own boiling temperature 110.0°C. This method is accurate and takes about 60 minutes.

(x) CALCIUM CARBIDE PRESSURE

Calcium carbide combines with water to form acetylene-gas and calcium hydroxide. This very principle is utilised in this method. Measured quantities of soil is taken from the rolled earthfill and mixed with powdered calcium carbide (also in a measured quantity) in a closed chamber. The pressure developed inside the chamber is directly related to amount of water entering into the reaction, which can be readily had from previously prepared graphs. This method takes as little as 8 minutes time and is accurate for most of the soils. In case of heavy clay soils, the result comes to be accurate, if some device is developed to break the clay-lump to pieces and enable entrained water in the soil to enter into reaction fully. With such devices, the method has been successful on clay soils, having P.I. Values as great as 40 even.

(xi) CALCIUM CARBIDE 'LOSS IN WEIGHT'

This method differs from that stated in (x) above in as much as the gas generated in the reaction is allowed to escape and the moisture content is determined from the loss in weight of the total mixture, as a result of chemical reaction.

(xii) PYCNOMETER

In this method sp. gravity of wet soil sample is determined and compared with that of the dry soil, previously determined in the laboratory. It takes 10 minutes time and is practical for field use, but requires proper identification of the soil at field.

(xiii) PRESSURE PYCNOMETER

The Texas method(31) eliminates error due to bubbles and froth in the pycnometer by applying 1200 p.s.i. to the system

(xiv) ALCOHOL SOLUTION

Following steps are involved in this method

(1) Fixed amounts of alcohol and damp soil are mixed together

(2) Sp. gravity of the alcohol-water solution is measured with a hydrometer

(3) Corresponding moisture content is read against (2) from a chart relating sp. gravity of alcohol-water solution to percentage of water. This method takes 8 - 10 minutes time and has been found to be sufficiently accurate for field work.

(xv) REFRACTIVE INDEX

In this method, chart relating refractive index of dioxan-water solutions to percentage water content is kept ready at site. To measure the percentage moisture content of rolled earthfill, set amount of Dioxan and damp soil are taken and filtered off to have a few drops of Dioxan water solution. Refractive index of the dioxan-water solution, having been measured, the percentage moisture content can be read from the chart.

(xvi) ELECTRICAL RESISTIVITY AND ELECTRICAL CAPACITANCE [illegible]

As their name suggest, physical properties like electricity resistivity and electrical capacitance of the

In the first two methods compacted soil fill are compared with those of similar soil having different moisture content, and plotted in a graph earlier. The more compacted and denser the soil, the more it will require electrical resistance for passage of electrical current and thus it gives an indirect measure of moisture content in the soil fill duly compacted.

Turnbull Drop Test is similar to Proctor's Penetration Test and moisture content of soil fill is estimated against the resistance off rod in penetration through the compacted soil fill.

Sulphuric acid combines instantly with water accompanied by temperature rise. The extent of temperature rise gives a fair idea of water being present in the soil.

(xvii) EMPIRICAL METHOD

This method utilises two quadratic equations to determine moisture content of a soil fill[8]

(i) $$\frac{W}{D_m} - \frac{1}{G} = \frac{M_o}{100} + \frac{1}{10\sqrt{C}}$$

(ii) $$\frac{W}{D_m} - \frac{1}{G} = \frac{M_o}{100}\left(1 - \frac{W}{D_w}\right) + \frac{1}{10\sqrt{C} + C(M - M_o)}$$

Where W = Density of water

D_m = Maximum dry unit weight

G = Sp. Gravity of soil

M_o = Optimum water content

C = A constant

D_w = Wet unit weight of soil fill

M = Field water content

Above two quadratic equations can be solved for two unknowns n and K_p when others are known. Standard values (or pre-determined values at laboratory) for K, D_w, G and K_e are taken for putting in the two equations and D_f determined at site about 1n/30 minutes time, by methods explained in Article 3.9.2.(D). Total time taken may be about an hour.

3.3.8. METHODS FOR DETERMINING UNIT WEIGHT

Different methods are explained below. A summary of such methods is also given at Table No.3.8. Primarily, methods can be classified as

(i) Disturbed Methods

(ii) Undisturbed Methods

(iii) Other Methods (New); Limitations useful precautions of the methods and their merits and demerits have been discussed below.

(i) DISTURBED METHODS

(a) SAND CONE METHOD

Weight of undisturbed samples is taken in usual way and inplace values of the removed soil determined by this method. The unit weight is governed by following factors:-

(1) Height from which sand is poured.

(2) Vibration present

(3) Moisture content of the sand

(4) Temperature

(5) Foreign material with sand

Following precautions are necessary to ensure accurate results:-

(1) Standard means for depositing sand in the test hole that will be uniform for different operations.

TABLE No.3.2

OIL.

Characteristics	OTHER NEW METHODS					
	r e	Hilf's Method (Designation E-25 Of USBR Earth Manual	Volume meter (Bs 1377 1948 Test No. 108	Poroso meter (used extensively by B.I.H.A.R Khagaul)	Washington Densometer & Cindo Soil Test Volumeasure	K.L. Rao Method
(1)		(8)	(9)	(10)	(11)	(12)
(1) Speed	ast utes)	Fast to very fast	More than 60 mts	More than 60 mts	Fast to very fast	Fast (20 mim
		moderately accurate for field purposes		2. Lengthy process	used in India	O.M.C. & soil is homogeneous
		3. Gives moisture contents nearer to actuals if field moisture content is above O.M.C. & soil is homogeneous				2. But Moisture content is obtained later on for finding deviation of field moisture content from O.M.C.
8) References		9,10,25	8	8	8,36	45

TABLE No.3.2

CHARACTERISTICS OF DIFFERENT TEST METHODS FOR DETERMINING IN-PLACE UNIT WEIGHT OF SOIL

Characteristics	DISTURBED METHODS IN WHICH TEST HOLE VOLUME IS MEASURED BY			Undisturbed Method in which Sample is Removed as a		OTHER NEW METHODS					
	Sand Cone (Designated E-9 of U.S.B.R.)	Oil Replacement	Water Balloon	Drive Sampler	Block Sample	Nuclear (Surface Type)	Hilf's Method (Designation E-25 Of USBR Earth Manual	Volume meter (BS 1377 1948 Test No. 108	Poroso meter (used extensively by B.I.H.A.R Khagaul)	Washing-ton Densometer & Cinco Soil Test Volumeasure	K.L. Rao Method
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
(1) Speed	Moderate (45 minutes)	Moderate (45 minutes)	Moderate to fast less than 45 minutes	Fast to very fast (30 minutes)	Slow	Very fast (10 minutes)	Fast to very fast (30 minutes)	More than 60 mts	More than 60 mts	Fast to very fast (20-30 minutes)	Fast (20 minutes)
(2) Stability with[a]											
(a) Granular material without cohesion	No	No	No	No	No	Yes	No			No	No
(b) Granular material with cohesion	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
(c) Fine grained cohesive soil	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
(3) Applicability to initial readings on rough granular surfaces	Initial reading possible on cohesive material;	Initial reading not practical	Initial reading simple	Not Necessary	Not Necessary	Rough Surfaces levelled with sand	Not Necessary			Initial reading simple	Not Necessary
(4) Precaution necessary	1. Use clean dry sand 2. Calibrate sand often 3. Avoid vibration 4. Use large dia. holes in material with coarse aggregates	1. Use level smooth test-site 2. Do not use with open graded or granular material in which oil might be lost by seepage	1. Use large dia. hole in material with coarse aggregate 2. Apply sufficient pressure to water to force balloon to fit test hole, but not distort hole	1. Use 3"-4" dia. sample in very dry sand and coarse grained soil 2. Insert sampler Uniformly fast	1. Donot use unless sample can removed intact	1. Should be used only by sufficiently trained operator	1. Accuracy in interpolating corrections etc. from graphs and reading other calibration curves	-	-	As with water balloon method	-

(2) Use clean dry sand that is uniform in size distribution.

(3) Calibrate sand frequently to determine its weight/cft under varying temperature and humidity conditions. A minimum of twice a day of such calibration is recommended.

(4) Use large diameter test holes, if material is mixed with large aggregates to minimize inability of the sand to fill large cavities in the sides of the holes.

(5) Prevent any vibration from settling the sand in the test holes during measurement.

(6) Do not reuse sand, contaminated with soil or water.

(7) Do not use this method with very wet pervious material in which bulking of sand due to excessive moisture content is likely.

(D) OIL REPLACEMENT METHOD

Weight of disturbed soil sample is taken in usual way and the inplace volume of removed soil is determined by oil replacement. Determination of test-hole-volume by this method is limited by

(1) Permeability of the compacted soil to allow loss due to penetration through it and the ability of the operator to level the test site

(2) Variation in the unit weight of the soil due to temperature change, and

(3) Amount of contained air in the soil

Following precautions are desirable to obtain accurate results.

(1) Calibrate the oil in a container of the same shape and approximate volume as the test hole. Pour the oil into the container at the same rate as used in the field.

(2) Level the test site so that the excavated hole can be poured brimful of oil. A bar level can be used as a guide.

(3) Pour the oil quickly into the hole to minimize the tendency for loss by seepage and to allow the same small amount of time for dissipation of air bubbles formed in the oil during pouring as was allowed during calibration of the apparatus.

(4) Do not use with dry or open graded aggregate in which oil might be lost by seepage.

(5) Use free flowing oil : SAE 20 to 30 in warm weather, SAE 10 weight in cold weather.(SAE - Society of Automobile Engineers 485 Lexington Avenue, New York)

(c) WATER BALLOON METHOD

In place volume of removed soil is determined by volume of water balloon in this method. The accuracy of the water balloon is limited mainly by the ability of the balloon to fit the hole. This in turn is influenced by the stiffness of the balloon, the fluid pressure inside the balloon, the shape of the hole, and the quantity of air trapped between the balloon and the sides of the hole. The following precautions should be taken to ensure accurate results-

(1) Use appropriate size device so that balloon is about the size of the hole required.

(2) Use large diameter holes in material mixed with coarse aggregates to minimize error due to poor fit along sides.

(3) Use sufficient fluid pressure to force the balloon to fit the holes but not so much that the hole is distorted or the base plate lifted off the ground. A pressure of 3 to 7 p.s.i. has been recommended.

(d) Prevent entrapment of air between the balloon and the hole in impervious material.[83] Washington Densometer has developed technique to fill the balloon from below thereby flushing air out to surface.

(e) Balance the vertical pressure on the base plate against the fluid pressure to prevent covering the hole in or blowing the balloon out.

(ii) UNDISTURBED SAMPLES

(a) DRIVE SAMPLE METHOD

This method is suitable for fine grained soils clays, silt and fine sands. Drive sample is a short sampling tube which is required to be pushed inside the compacted earthfill and then taken out; unit weight of soil sample is determined by dividing the net weight of the sample thus obtained by volume of the tube. Its accuracy is limited by the amount of compaction or loosening of the sample while the sample is driven in. In general the unit weight of loose to medium dense soil tends to be increased and the unit weight of very dense soils tends to be decreased.[41]

(b) BLOCK SAMPLE METHOD

Method is only suitable to soils which can withstand its shape and do not flow out while sampling. The block of soil is coated with a known amount of paraffin, then weighed and then immersed in an overflow volumeter to determine volume of the sample. Unit weight of soil is then determined by dividing the weight of the sample by the volume of the sample.

(ii) NUCLEAR METHODS

(A) [illegible]

This method for determining moisture content and density of soil still is relatively new. A number of literatures (10, 16, 63) have dealt with it but so far most of the works have been confined to laboratory studies involving the development of instrument and some small scale experiments in the field. Underlying principle is explained below in a few words -

(a) [illegible]

If a source of 'fast' neutrons is placed in a soil, neutrons will scatter in a random manner from the source through the soil. These neutrons are particles of matter having a mass approximately equal to that of a hydrogen atom and having high kinetic energy. As they enter through the soil, they collide with atoms in the soil and rebound as objects do in an elastic collision. The neutrons loose some of their energy in these collisions. Since the hydrogen atom is one atom in most soils with a mass nearly equal to that of neutron, the neutrons which are scattered will loose more energy in collisions with hydrogen atoms (atomic weight 1.008) than they will in colliding with other atoms commonly found in soils (Oxygen, atomic weight 16,000). Some of the neutrons will return to the vicinity of the source after many collisions at a much lower energy level. If a counter, capable of recording these slow neutrons is kept in close proximity of the source of radiation, a count of neutrons may be obtained which will reflect the quantity of hydrogen atoms in the soil. Fortunately, the principal source of hydrogen atoms in a soil is water and therefore, such a count may be used to indicate the amount of water in the soil surrounding the source of 'fast' neutrons.

(D) DENSITY DETERMINATION USING GAMMA RAYS

If a source of Gamma radiation is placed in a soil, gamma rays will be emitted from the source into the surrounding soil. These rays will be scattered by the 'electrons' in the soil and loose energy in the process. Some of the scattered rays will return to a detector near the source and can be counted. The number of Gamma rays counted will depend upon the average length of their path to the detector and their energy when they reach the detector. By properly shielding the gamma ray detector from the source of gamma radiation and placing the detector a specific distance from the source, an experimental curve can be drawn which will reflect the density of most common soils in terms of gamma rays counted during a time interval. For the common types of soil in which the engineer is interested, a low count gamma rays will indicate high density and a high count low density. As the density of a soil increases, the electron density increases proportionately and causes greater scattering and energy loss of the gamma rays. Thus the chance that gamma rays will be scattered back to be counted becomes smaller and so the count rate drops.

Out of various tests using nuclear device for observing field compaction, following conclusions have been drawn, which are quite encouraging -

(1) The nuclear method can be used for the field control provided the depth at which measurements are made is not less than 6 inches.

(2) Calibration curves developed on the job yield results in closer agreement with the moisture and density measurements by sampling than did the use of calibration curves, developed in the laboratory. This is particularly true for density.

(3) The moisture contents by the nuclear procedure were in fairly close agreement with the sampling procedure, the average of all measurements by the two procedures being within one percent (dry weight) of each other.

(4) The densities obtained by the nuclear procedure, however, do not compare as favourably with the densities by sampling as do the moisture contents, the average of each measurements for densities by the two procedures (using field calibration charts for the nuclear procedure) usually differ within a range of about 5 lbs.

There are two types of nuclear devices - the 'probe' which is designed to be lowered into the ground, and the other the 'surface gauge'. Moisture and unit weight probes are lowered to the desired depth in the soil through access tube driven into the ground and are particularly suited for making measurements at various depths and for making repeated or periodic measurements at the same points. Surface gauges are placed on the ground surface and ~~are~~ can be chiefly used in compaction control.

ADVANTAGES OF NUCLEAR METHODS

Following are advantages of Nuclear Methods over old conventional method.

(1) Inplace unit weight tests can be run in about one fifth of the time required for conventional disturbed or undisturbed methods.

(2) Soil need not be disturbed from its place.

DISADVANTAGES OF NUCLEAR METHODS

Disadvantages of Nuclear Methods can be listed as below

(1) Highly initial cost and uncertain accuracy.

(2) Claims of accuracy in unit weight measurements vary from plus or minus 0.5 to 6.5 lb/cft. It is however, true that their variation might also partly due to inaccuracies in the conventional methods, and the differences in sample size or samples tested.

LIMITATIONS OF NUCLEAR METHODS

Accuracy of current surface gauges is limited by following features:-

(1) Moisture determinations are influenced by hydrogen atoms in certain clays that are not normally driven off at 110°C, therefore, the gauge indicates higher moisture contents in some materials than the standard method does.

(2) Unit weight determinations are affected by the chemical composition of the materials being tested. Iron oxide, for example, would tend to make the unit weight measurements low, water in the soil tends to make them high.

(3) The operator cannot control the depth and/or volume of soil being tested for moisture content and unit weight. The

one. It also depends on the dimension of the gauge and the moisture content and unit weight of the soil. In unit weight tests, thickness of the soil sample decreases as unit weight increases. Maximum depth of sample with current equipment varies from 8 to 10 inches.(27)

(4) Unit weight determinations are sensitive to air gaps between the gauge and the ground surface. Tests indicate that a 1/100 in. air gap will reduce the measured unit 1 lb/cft.

(5) Other sources of error include the non-uniform distribution of the radio-active material, reflection from nearby objects of stray radiation that escapes through the top and sides of the gauge and variability in counting time.

While working on nuclear devices, operating personnel might be endangered to face excessive radiation exposures. Hence, steps must be taken to have safeguards against this.

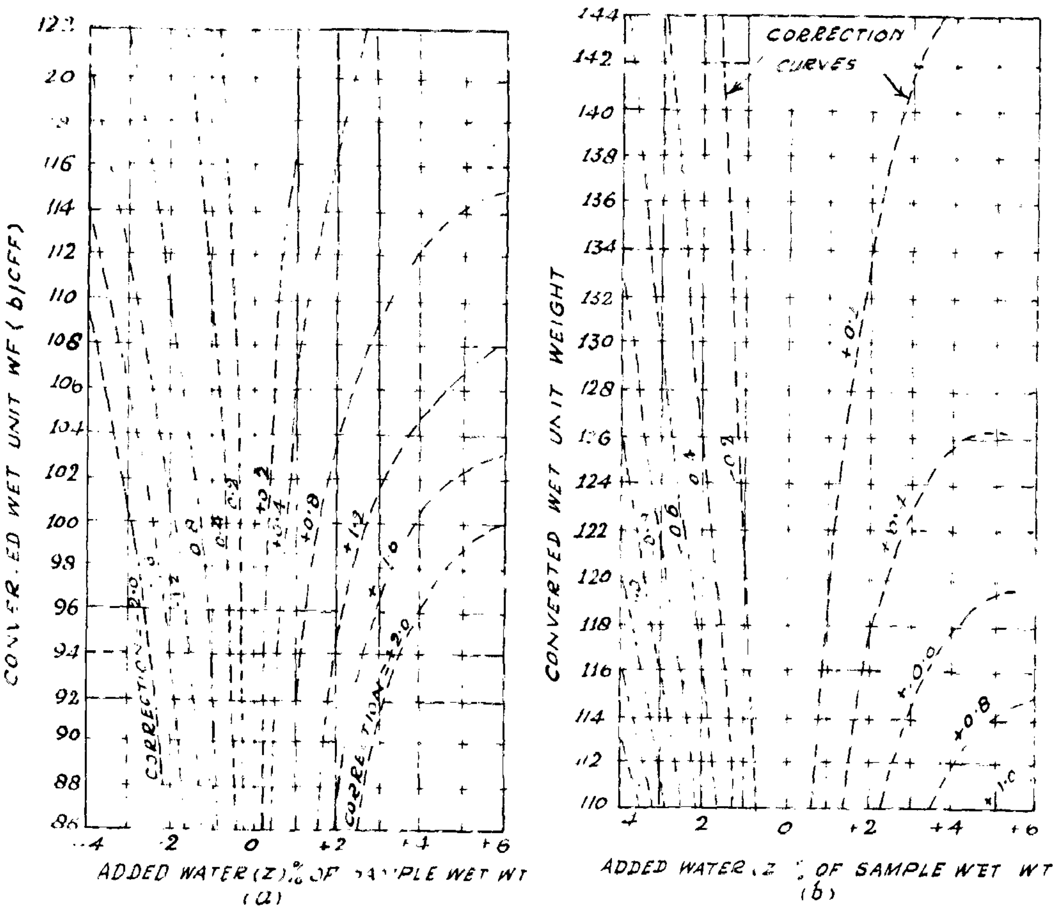
(D) [illegible]

(a) DETERMINATION OF MAXIMUM DRY UNIT WEIGHT OF SOIL FILL

Hilf(28) has evolved a quick method to determine maximum dry unit weight of soil fill without making it obligatory to know the actual percentage water content and this can be utilized to pass a soil fill layer at site. Maximum dry unit weight of particular soil will have to be determined earlier from laboratory tests. Steps involved in this method are summarized below:-

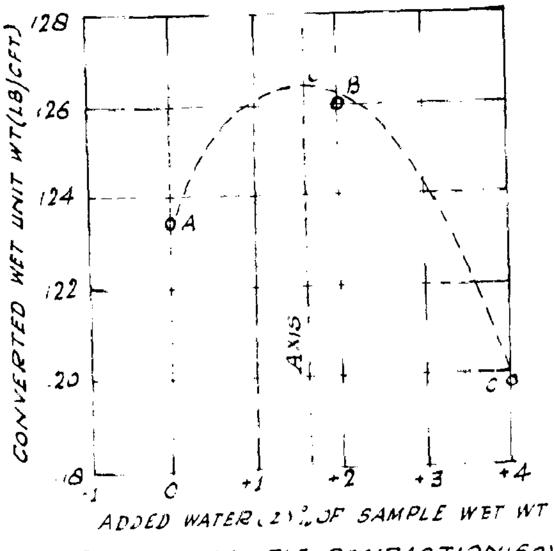
(1) Determine field unit weight (or wet soil density) of rolled fill in any manner stated in this Chapter earlier.

(2) Plot a compaction curve (Fig.3.3) for which following steps will be followed:-



PLOT OF CORRECTION FOR USE IN CALCULATING DIFFERENCE FROM OPTIMUM WATER CONTENT FOR SARTS WITH CONNECTED WET DENSITIES IN THE (a) 86 - 122 LB/CFT AND (b) 110 - 144 LB/CFL)

FIG 3.4



PLOT OF SAMPLE COMPACTION (50) RESULTS

FIG 3.3

(a) Compact a sample of in-place material passing above No.4 at field water content in a Proctor's mould and obtain the wet unit weight. Plot this point as A on the "0" percentage vertical line in Fig. 3.3.

(b) Take 7.0 lbs sample of in-place material at field water content, add 0.15 lbs of water (2 % of wet weight of 7.0 lbs sample or 60 c.c. water). Compact, determine the wet unit weight, divide this value by 1.02 to obtain the converted wet unit weight and plot as point B on the + 2 % vertical in Fig. 3.3 Converted wet unit weight is the wet unit weight divided by 1 + Z where

$$Z = \frac{\text{weight of water added to sample}}{\text{wet weight of sample}}$$

(c)(i) If point B has a greater unit weight than A, take a 7.0 lbs sample of in-place material at field water content add 0.30 lb of water (4 % of 7.0 lbs sample or 130 c.c. of water), Compact, determine wet unit weight, divide by 1.04 to get the converted unit weight and plot as point C on the +4 % vertical line in Fig. 3.3.

(ii) If point B is less than A but more than about 3 lbs/cft, take a 7.0 lbs sample of in-place material, let it dry to about 2 percent (2 % of 7.0 lbs) compact, determine wet unit weight and divide by 1-Z where Z is the amount of water lost in drying. (If 2 % of the wet unit weight were lost, divide by 1.00 = 0.03 or 0.00). Plot converted wet unit weight as point C on the vertical line corresponding to 2, the amount of water lost in percent which is 2 % in this case, shown at Fig.3.2.

(iii) If point B is less than A, but within 3 lbs/cft take a 7.3 lbs sample of in-place material add 0.073 lbs of water (1 % of 7.3 lbs equal to 33 C.C. water), compact, determine wet unit weight, divide by 1.01 to get the converted wet unit weight and plot as point C on the +1 vertical line.

Fit a parabola through the three points A,B,C free-hand or through any known theoretical method. One such method is given as Appendix 3.2 of this Chapter.

(3) Find out peak of the parabola, which gives maximum converted wet unit weight of the soil (120.6 lb/cft in this case) Fig. 3.4

Hence, % Maximum Dry unit weight (Wd/Wo) = $\dfrac{\text{Average wet unit of soil}}{\text{Maximum converted wet unit weight, as obtained in step (3) above}} \times 100$

Field Engineer can compare this figure with allowable compaction factor for the Project, say (95 %) and decide whether to pass a compacted fill layer or not.

(D) <u>DETERMINATION OF % MOISTURE CONTENT OF SOIL FILL</u>

Above graphs need to earlier plotted in laboratory to apply corrections, as stated below:-

(2) Find the % of water required to be added to get the maximum converted wet unit weight of the wet soil. From Fig. 3.3, this is 1.0 % for example in this very case, and can be stated to be peak point (0)

(3) Required correction is got from Fig. 3.4 (a) & (b) by plotting peak point (0) on the correction graphs in the Fig. 3.4. This is obtained as +0.8 in this very case against 1.0 % peak point and 120.4 lb/cft as wet unit weight. The difference between optimum water content and in-place water content is then equal to 1.0 + 0.8 or 1.8 %. The plus sign indicates % water to be added in the field to obtain optimum water content. Hence, Variation of fill moisture from optimum ($w_o - w_f$) = 1.8 %. This is a sufficient guide to decide whether the fill moisture content is within tolerable limits (viz. ± 2 % of O.M.C.) or not. If at all, exact amount of further water to be added to the fill is to be determined to make it tally with O.M.C its exact moisture content (w_f) will have to be determined by any methods of moisture determination described earlier.

CONCLUSION

(a) Above method is pretty fast (50 - 60 minutes) to know

(1) Exact percentage of standard maximum dry unit weight = $\frac{\text{Maximum wet unit weight}}{\text{Maximum converted wet unit weight}} \times 100$ (W_d/W_o)

(2) Approximate difference between optimum water content and the in-place water content ($W_o - W_f$)

(b) It will however, be essential to know, in-place water content (which may take another 18 hours to 10 minutes by different methods as discussed in Article 3.3.2.(A) earlier to determine following informations for routine record at site.

(1) Field dry unit weight

(2) Moulded dry unit weight at inplace water content

(3) Standard maximum dry unit weight

(4) Optimum water content

Informations so obtained in B(a) above are sufficient to accept or reject a rolled fill layer quickly and that is why it has become comparatively popular as seen. U.S.B.R. have been following it all along with success and to use it mechanically and reduce time further they have published compact graphs. Such graphs and methods of using them are given in References(9,10) and can be readily referred to for soils used as rolled earth fill. However, use of the graphs will be better appreciated after the above steps are understood correctly.

(C) WASHINGTON DENSOMETER

This method utilises a thin wall balloon which is designed to fill the hole from the bottom up and to fill small voids in the wall of the hole more easily. It is an improvement of the water balloon method, discussed earlier and takes about 20-30 minutes time.

(D) R.J. RAO'S METHOD

This method is the same as Hilf's method or U.S.B.R. method for determining compaction ratio (W_c/W_o). Rao has given mathematical interpretation of Hilf's solution as below, but he suggests repeating the Proctor's test on a single core by adding two or three moistures to it, instead of repeating the test with a number of cores taken from the rolled fill and then adding moisture in different proportions to get wet density or converted wet density curve, as outlined by Hilf, by other cores taken from the same strip used only their wet density to be determined and plotted on such curves drawn for the core to obtain ($w_o - w_f$) for the sample. Moreover compaction ratio (W_d/w_o) can be readily obtained in one operation by adding known moisture to soil and noting its wet unit weights before and after such addition of moisture.

MATHEMATICAL INTERPRETATION OF HILF'S METHOD

ASSUMPTIONS

As long as soil specimen is not saturated there is no change in volume when a compacted specimen dries up. Take soil of one gm dry weight, let

m = moisture content

w_{f_1} = Its wet unit weight

w_d = Its dry unit weight

Then volume of one gm of dry soil $= \dfrac{1}{w_d}$

and moist weight = $(1 + m_1)$

$\therefore$ Wet unit weight $W_{f_1} = (1 + m_1)\, w_d$

$$\text{or} \quad (1 + m_1) = \frac{W_{f_1}}{w_d} \qquad \ldots \quad \text{Eqn (1)}$$

At O.M.C. suppose m_2 gm of water were added for every gm of wet soil. Take soil of one gm of dry weight

Let w_o = dry unit weight at O.M.C.

Dry Volume $= \dfrac{1}{w_o}$

One gm of soil (dry) becomes $(1 + m_1) + (1 + m_1)\, m_2$

$= (1 + m_1) + (1 + m_2)$

Wet unit weight of soil at O.M.C. (W_{f_2}) $= \dfrac{\text{Mass}}{\text{Volume}}$

$= (1 + m_1)(1 + m_2)\, W_o$

$$\text{or} \quad 1 + m_1 = W_{f_2}/\, w_o\, (1 + m_2) \qquad \ldots. \quad \text{Eqn. (2)}$$

Equating Eqn.(1) & (2)

$$\frac{W_{g1}}{V_d} = \frac{W_{g2}}{V_o} / (1 + m_3) \quad \text{Or}$$

Compaction Ratio $$\frac{V_d}{V_o} = \frac{W_{g1}(1 + m_3)}{W_{g2}}$$

3.2.6. [illegible]

(A) [illegible]

Electric Oven heaters with or without temperature control are designed to make the soil sample completely dry which is the only known method to determine exact moisture content of soil samples. However, they are quite time consuming and it is mainly on this account that, they are discarded for rapid construction control works. Further handicap in this case is electricity, because where electricity is still not available at site, electric oven or forced draft tube heaters cannot work. In case of open pan heating, another danger of the soil being overheated is to be guarded against. The latter method has, however, to be taken recourse when electric ovens cannot be made use of at site for want of electricity or when such ovens are not available.

Alcohol burning is simpler than oven heating in that temperature remains controlled near about 150°C - 200°C and there is no danger of over-heating the soil. But it takes comparable time as with ordinary oven heaters viz 30 minutes.

Proctor Penetration or Turnbull drop tests are more or less similar and give an indirect method of assessing fast, moisture content of soil samples. Probably this is the easiest method to assess moisture content provided reference graphs have been prepared earlier for the soil being used on embankment and no mistake is done in identifying the soil for which graphs are valid.

Hilf's method requires knowing difference of actual water content and O.M.C. of soil ($W_o - W_f$) instead of determining the actual water content; and this difference gives enough guide for a construction engineer to decide whether further water is to be added to the soil or deducted for compaction. This together with necessary 3 compaction of the soil takes about 60 minutes time. This method may be adopted for rapid control purposes unless, other new methods are developed, which give still quicker and accurate results.

Compacted wet unit weight is satisfactory (20 minutes) if the soil can be correctly identified and compaction curve for the particular soil is available from before hand. However, it does not give exact moisture content as in case of oven drying methods and is useful for homogeneous soils only. Where the borrow area is stratified and excavation is done by shovel unit, the method will give erroneous results.

Nuclear methods are quickest (10 minutes) and give fairly accurate results for field purposes. However, their use is restricted because of non-manufacture of the instruments on larger scale. Notable disadvantage to the method is the nuclear radiation which, in any case, has to be guarded while working with such instruments.

Use of Distillation Method is restricted at field laboratories, because it requires a course of running water to cool the condenser all along. In any case, it takes longer time (65 minutes) than Hilf's Method, compacted wet unit weight method or nuclear method and hence not so much helpful to rapid construction control.

Calcium carbide methods are useful for field in as much as it takes only 10 minutes time and gives fairly accurate results. Even with heavy clay content the method has been successful with special appliances, but sample size limits the accuracy of results in 'Calcium Carbide Pressure Method' while commercial apparatus remains yet to be manufactured for "Calcium Carbide Loss in Weight Method".

Pycnometer methods (10 minutes) are essentially laboratory methods, but they are practical for field use also. Proper identification of soil is necessary to compare specific gravity of the wet soil to that of the dry soil, previously determined in the laboratory.

Refractive Index Method is yet of academical importance because no steps have been taken to popularise it and make it of practical use.

However, since it takes "little time of 10-minutes in determining large moisture content and gives accurate results it can be tried on rapid construction control works with more confidence and ease.

Alcohol Solution Method taking 10 minutes time gives sufficiently accurate results for field works, as with other comparable methods like that of nuclear and refractive index devices.

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Electrical Resistivity and Electrical Capacitance Methods are not as accurate Methods but may be of some avail only where electric implements can be brought to site for use and experts are available to use it.

Temperature rise on addition of sulphuric acid, does not give exact knowledge of the soil moisture content, but a fair idea of water being present with soil. It can be used at site only with great caution to avoid any burn injury.

Empirical Methods involve mathematical steps and solving quadratic equation. It cannot therefore be called as a practical method. Besides involving mathematical calculations, sometimes it results in imaginary figures, difficult to explain. Moreover by increasing compactive effort, it is possible to get the same maximum dry unit weight at a lesser water content and in such case formula fails to provide a solution. The method also fails to give a solution when soils from different borrow area of different conditions and sp. gravity are brought and compacted at the site, together.

(D) UNIT WEIGHT DETERMINATION

(1) DENSITY IN SITU

Sand Cone Method is not very accurate and the results are likely to vary due to variability of unit weight of sand and its inability to completely fill test holes which is guided by its angle of repose. However, with due care, sufficiently accurate results can be got for field purposes. Oil replacement methods have also certain limitations elucidated earlier, but due to its fluidity it gives better results than the sand displacement method.

Water Balloon Method is as simple as other two, but has its own limitations. In coarse material, both sand cone and water balloon methods are likely to give erroneous results. While the oil replacement apparatus being quite a large apparatus than other two, is an obvious disadvantage. Another disadvantage with all disturbed methods is that weight of the samples taken out from the hole will have to be instantly taken at site to avoid loss of moisture content and thereby reducing wet weight of the soil. But unlike undisturbed methods, it has advantage of being used for coarse grained material besides fine grained ones.

(ii) UNDISTURBED METHODS

Drive sampler type can be used for determining wet unit weight of fine grained soils at site. However, its sample size is limited by sampling tubes. In Block sample type, sample size is not limited by any apparatus but cannot be correctly used unless block can be taken out intact from the site. Block sample process is slower than the drive sampler type, but gives better results in case of larger samples taken out intact. Undisturbed methods have advantage over the disturbed methods in that, samples can be brought to the field laboratory and weight determined conveniently than the same being taken at site itself. This is because, no appreciable loss of moisture is anticipated in short transit with drive sample type or block sample type specimens.

(iii) NUCLEAR METHODS

Nuclear method is the quickest method and gives fairly accurate results for field purposes. However, the nuclear apparatus is costly and requires ample improvement and manufacture in commercial scale for being used economically. It takes 10 minutes time only in determining wet unit weight of the soil sample, but requires the operator to be guarded from the effect of nuclear radiation.

Hilf's method takes 30-60 minutes time to determine % maximum dry unit weight of soil which is so very useful to control compaction of the soil fill. Unlike previous methods, it acquaints one with the % compaction of the soil instead of giving actual wet unit weight. Method is simple and moderately accurate. However it cannot be used for soil contained with gravel or rock.

Volumenometer and Penetrometer are simple to use but involves quite lengthy processes. Volumenometer and Penetrometer can be used in case of cohesive soils only. Penetrometers are not being manufactured yet and has remained so far limited to laboratory uses. Washington Densometer and Cenco Soil Volumenometer are improvement over water balloon method, in that the balloon is inflattened from below to upwards, driving all entrapped air along the side walls out, andhence gives better results than the former. K.L. Rao's simplified formula for Hilf's Method gives compaction factor near to actuals if field moisture content is below O.M.C. and soils is homogeneous. It has been established further that the original Hilf's method gives compaction factor closer to actuals if field moisture content is above O.M.C.(88) However, since there is not much variation in the compaction factor given by two methods, it is advisable to use K.L. Rao's Method in preference to Hilf's method, the former having advantage of determining compaction factor easily and quickly. It helps to control long lengths of embankments with less staff etc. Main different between two processes lies in the fact that Hilf's Method whereas primarily necessitates repeating of proctor's tests with a number of cores in one strip of rolled fill, Rao's method involves repeating proctor's test with one single core by addition of two or three minutes and determining compaction factor, by simplified formula of Dr. K.L. Rao.

In a nutshell it can be stated that though different methods for determining moisture content and density of compacted soil fill have been evolved, none is universally accepted. All such methods have their own limitations and different countries have been using one or the other method as per their convenience and availability of resources. Of course efforts have been made to improve results by taking precautions. In India even now conventional methods of oven drying for moisture content determination and disturbed and undisturbed methods for wet density determination of soil are very common. Rao's method has been shown to be quicker[68] than Hilf's Method and still former giving results very fairly tallying with that of the latter. In this context future of Rao's Method appears to be promising. In future, when other quicker methods as Nuclear Method, Refractive Index Method etc. are perfected and their apparatuses manufactured on commercial scale, they will prove to be more useful for rapid construction control works.

APPENDIX - 9.1

SWEDISH METHOD OF FIELD PERMEABILITY FOR PASSIVE FILL LAYERS

At Goschenenalp Dam (Switzerland) either circular or Trapezoidal pits were made and water was passed into them to measure permeability through compacted fill of the embankment. Permeability tests were conducted by constant head and variable head methods using one of the formulae as below:-

(i) Constant Head Method in a circular hole

$$K = \frac{4}{30.6\,h} \times \frac{4Q}{d^2}$$

Where
- d = dia of hole
- h = head of water
- Q = Constant discharge
- K = Coefficient permeability in cusec/sqm

(ii) Falling head method in a circular hole of dia. d

$$K = \frac{d}{30.6\,h_m} \times \frac{\Delta h}{\Delta t}$$

Where
- Δh = Fall of head in Δt
- h_m = Mean head of water

(iii) Constant head method in a Trapezoidal hole with a square base of length b

$$K = \frac{1}{27\sqrt{\frac{h}{b}} + 3}$$

Where h = head

(iv) Falling head Method in Trapezoidal hole of base b

$$K = \frac{\left(1 + \frac{8\,h_m}{b}\right)^2}{37\,\frac{h_m}{b} + 3} \times \frac{\Delta h}{\Delta t}$$

APPENDIX - 3.2

FITTING A PARABOLA THROUGH POINTS A, B, C OF FIG.3.5

If more than three points are available, three points closest to optimum are used.

CASE I

When B is above A and C, following construction steps will be followed:- (Vide Fig.3.5 & 3.6)

(1) Draw horizontal base line through A say AK

(2) Draw vertical lines through B and C as BM & CL
D is intersection point of AK & BM

(3) Join AB and draw DE parallel to AB, E being intercept on vertical through C

(4) Draw horizontal line EF to have an intercept point F on BM. (F will coincide with D if A, B, C are equally spaced horizontally)

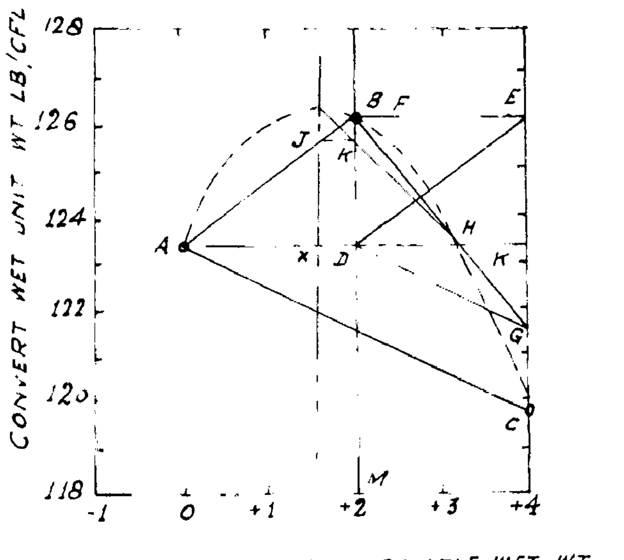
(5) Join AC and Draw FG parallel to AC to establish point G

(6) Join DG (if they do not coincide as explained above) and find out point H at base line AK. This will be a point on parabola.

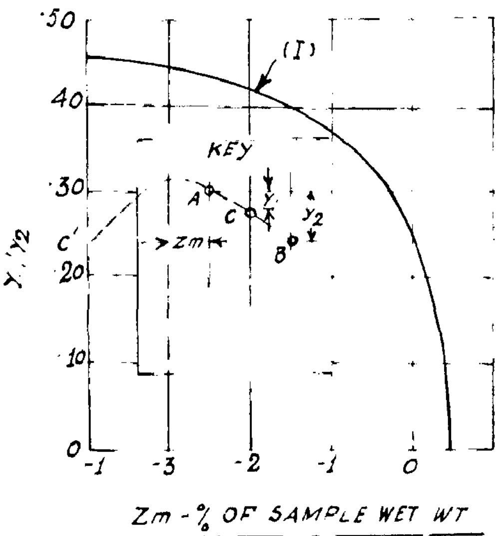
(7) Divide AH at point 'X' draw a vertical through X, OX is axis of required parabola.

(8) Find point J at intersection of AB and OX. Drop JK perpendicular to BM. Join KH and extend it to cut OX at O. O is the peak point of parabola

(9) Draw parabola through, A, O, B, H and C (use [illegible] points)



CURVE FITTING
FIG 3.5



CURVE FITTING
FIG 3.6

CASE IX

When A,B,C are within 3 lbs/sft of each other and left point (A) is highest, following preliminary steps will be followed before following above steps:-

(1) Calculate V_1/V_2 where $V_1 = A - C$ and $V_2 = A - B$ (see Key of Fig.3.6)

(2) Determine the horizontal distance 'Z_m' between point A and the axis of the parabola from the curve (X) in Fig.3.6.

(3) Plot the mirror images of points A, B, and C on the left side of the axis point as A_1, B_1 & C_1

(4) Redesignate point C' as A, point A' as D and point C as B and follow steps 1 to 9 as outlined in case 7 above.

CHAPTER - 4

CONTROLLING COMPACTION EFFORT

Controlling compactive effort is important for economical and better compaction of earth dams. It depends upon correct choice of 'compaction method' and 'compaction machines' commensurate with needs of a particular soil.

4.1. COMPACTION METHODS

Compaction of fill materials of an high earth dam will depend upon its characteristic materials to be used. Such material can be classified as

(a) Hard rock

(b) Soft rock

(c) Gravel and sand, with or without silt and clay

(d) Clay, silt and predominantly clayey soils

Of the four classifications of construction materials stated above, former two can be termed as rockfill materials while the other as earthfill materials.

(A) 'Rockfill' can be compacted by

(a) Rolling and sluicing

(b) Vibrating and sluicing

(c) Independent sluicing by high pressure jets.

In some Algerian Dams, even manual labour aided by cranes have been successfully employed for setting heavy stones of the rockfill in position. Others in particular U.S.A. have advocated for high lift construction of rockfill for sake of speed of construction. In this method, rock is dumped from a top elevation and its weak and friable parts are allowed to break away. Such break-away fines are washed by heavy water jets called 'sluicing'.

In San Gabrial Dam No.2, they allowed only dumping of rocks from a top elevation, with a little of sluicing. This cannot be said to have been compacted and the result was obvious, the Dam settled by 12 ft, only in the next rains. So, if 'sluicing' is adopted for compacting earthfill, it should be used profusely to ensure that all fines have been washed off. In Hirfanly Dam (Turkey) this method of 'dumping and sluicing' was successfully adopted.

In European countries, compacting rockfill is preferred by rolling and vibrating. A recent example of this class is Quoich Dam (126 ft high) in Scotland; Efficacy of compaction by vibrating is however doubted in some quarters. While British Engineers claimed its success in construction of Quoich Dam[73] Zeller and Zeindler[74] were not very much impressed by its performance on Geschenenalp Dam (Switzerland). It appears, compaction by vibrating in latter case failed because of having adopted thick layers of rockfill for compaction. At Geschenenalp Dam, the dry density of the fill dumped in 6 to 12 ft thick lifts did not increase substantially after vibrating with 15 tons vibrators. In Quoich Dam, however, the method was successful in rolling comparatively thin layers - 2' thick rockfill, first by 10 tons smooth roller and then by vibrating. Judged as a whole dumping and sluicing of rockfill in high lifts is considered more economical and quicker than other compaction methods provided other peculiar conditions at site do not outweigh it.

In soft rock compaction by steel tyred vibration has been seen to be quite effective.[7]

(B) Earth compaction principally includes compaction of

(i) Cohesionless gravel and sand on the one hand, and

(ii) Cohesive clayey soils on the other

Other classes of earthfill lie between these two extremes. While quite a variety of compaction implements have been used in individual cases and by different agencies, compaction of the former class is usually preferred by

(a) Rolling

or (b) Vibration

while the latter is compacted by

(a) Rolling (Plain Rollers)

or (b) Tamping (Sheep foot rollers)

In cases of cohesionless gravel or sand layers extending in deep extremes, internal deep vibrators (Vibroflotation)[71] Air Injection[72] or explosive blasts can be as well utilised for effective compaction. In cohesive soils, however, conventional compaction by rollers are more suited.

6.8. CHOICE OF COMPACTION MACHINES FOR EARTHFILL COMPACTION

Selection of compaction method needs a thorough study of the materials to be compacted, economics of different compaction implements, and their availability. Selection of the most suitable type of equipment for an actual work also depends on other factors as placement moisture content, meteorological conditions etc. (Vide Table No.6.1)

6.8.1. MATERIAL

K.L. Rao has very broadly classified (vide Table No.6.8) the Indian Soils based on rainfall intensity by correlating it with soil density of the area[73]. Though the classification does not apply to all areas alike, on world map, even in India, it yet

S.No.	Name of th...	...oratory Values O.M.C. (%)	Maximum density (lb/cft)	Field Values Achieved O.M.C. (%)	Maximum Density (lb/cft)	Remarks
(1)	(2...	(9)	(10)	(11)	(12)	(13)
1.	Ukai	22 20	121 126		Not less than 98% Not less than 90%	Under Construction
2.	Hirakud	16 19	120 124	10 12	120-130 130-140	Completed (1956)
3.	Maithon	12.3 12.9	116 118			Completed (1956)
4.	Kolar	23 20	119 120			
5.	Vannadh...	16 13	120 121			
6.	Tenughat	16-19 13	110-112 116-120	+ 1 % - of O.M.C	Not less than 98% Not less than 98%	Under Construction
7.	Panchet	13 12.9	116 117			Completed (1959)
8.	Kotah B...	17 14	122 124			Completed (1957)
9.	Lower B...	- -	- -	18 %	123	Completed (1963)
10.	Mandira	16 13	120 121			
11.	... (Left Fla...	18 13	120 123			Recently Completed (1967)
12.	Vaigai	16-19 19	100 120			

TABLE No. 4.1

A COMPARATIVE STUDY OF [illegible] ... 1962

S.No.	Name of the Project	State	Maximum height of earth dam (ft)	Type	Slopes: Upstream	Slopes: Downstream	Zone	Laboratory Values: O.M.C. (%)	Laboratory Values: Maximum density (lb/cft)	Field Values Achieved: O.M.C. (%)	Field Values Achieved: Maximum Density (lb/cft)	Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1.	Ukai	Gujarat	280	Zoned	3:1 top 150' & 4:1 below	2:1	(a) Impervious (b) Semipervious (Silt 50-60 %)	28 20	121 124		Not less than 95% Not less than 90%	Under Construction
2.	Hirakud	Orissa	198	Zoned	3:1	2:1	(a) Impervious (b) Semipervious	10 15	120 134	18 18	120-130 130-140	Completed (1960)
3.	Maithon (D.V.C)	Bihar	164	Zoned	3:1 top 10' 3:1 below	3:1 top 10' & 2.25:1 below	(a) Impervious (b) Semipervious	18.3 18.0	116 118			Completed (1956)
4.	Kolar	Madhya Pradesh	150	Zoned	3:1	2:1	(a) Impervious (b) Semipervious	28 20	110 120			
5.	Vamsadhara	Andhra Pradesh	169	Zoned	3:1	2:1	(a) Impervious (b) Semipervious	10 13	120 131			
6.	Tenughat	Bihar	163	Zoned			(a) Impervious (b) Semipervious	16-10 18	120-118 116-120	+ 1 % - 2 % O.M.C	Not less than 95% Not less than 95%	Under Construction
7.	Panchet Hill (D.V.C)	West Bengal	153	Zoned	3:1	2.25:1	(a) Impervious (b) Semipervious	19 18.0	116 117			Completed (1960)
8.	Kotah Barrage	Rajasthan	120	Zoned	3:1	2.5 to 1 3:1 & 4:1	(a) Impervious (b) Semipervious	17 16	122 124			Completed (1967)
9.	Lower Bhavani	Madras	100	Zoned	3:1 top 20' & 4:1 below	2:1 top 70' 2.5:1 next 60' & 3:1 below	(a) Impervious (b) Semipervious	- -	- -	18 %	123	Completed (1953)
10.	Mandira	Orissa	90	Zoned	3:1	2:1	(a) Impervious (b) Semipervious	10 13	120 131			
11.	Nagarjunasagar (Left Flank)	Andhra Pradesh	[illegible]	Zoned	3:1	2:1	(a) Impervious (b) Semipervious	15 13	130 133			Recently Completed (1967)
12.	Vaigai	Madras	88	Zoned	2:1, 2.5:1 3:1 & 4:1	2:1	(a) Impervious (b) Semipervious	16-18 19	100 120			

TABLE NO.4.1 -(Contd)-

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
13.	Dudhwa	Madhya Pradesh	82	Zoned	3:1	2:1	(a)Impervious (b)Semipervious	14 9	120 124			
14.	Ahraura	Uttar Pradesh	75	-	-	-	-	-	114	12 to 14	92.5 to 109	Completed(195? (Dam failed in June 1953 - since Rebuilt)
15.	Tungabhadra	Mysore	65	Zoned	2:1 top 29' 3:1 below	2:1 top 20' 3:1 next 30' 4:1 below	-	116	134	-	-	Completed
16.	Saprar	Uttar Pradesh	60	Zoned	2:1	4:1	-	14	110	-	109	Completed
17.	Lalitpur	Uttar Pradesh	54	Homogeneous	3:1	4:1	-	16	113	-	104	Completed
18.	Naswa	Uttar Pradesh	52	-do-	3:1	2:1	-	18	118.5	-	96	Completed
19.	Nanak Sagar Dam	Uttar Pradesh	54	Zoned	3:1	2.5:1	(a)Impervious (b)Semipervious	18 15	100 107	19 16	103 108	Completed (1962)
20.	Moosakhand Dam	Uttar Pradesh	98	Homogeneous	3:1	2.75:1	-	14 $	112	11	114	Completed (1965)

remains to be testified on a broader rational survey of soils in different regions, and see whether Rao's assertions are right.

TABLE NO.4.2

TABLE BASED ON K.L. RAO'S FINDINGS

Average Yearly Rainfall	Density of Available Soil	Remarks
(1) 100"-300"	Maximum density of available material 50-75 lbs/cft	Idea of constructing "Sharavathy Dam"(Mysore) as earth dam in this region had to be abondoned because of low density of foundation material and masonry dam constructed, founded on sound rock at 100' depth
(2) 50" - 100"	Dry density of available material (a) Impervious soil - 112 lbs/cft (b) Shell material - 116 lbs/cft A little difference of 4 lbs/cft between impervious clay material and shell material has to be	In this region following dams have been constructed earlier. (a) Hirakud - 195' high (Orissa) (b) Maithon - 164' high (Bihar) (c) Panchet - 135' high ~~(Bihar)~~ (West Bengal)
(3) Less than 50"	Dry density of available material (a) Impervious soil - 105-108 lbs/cft (b) Semipervious soil 120 lbs/cft	Lower Bhavani Dam (Madras) 95' high is located in such region

From above it appears that soils, falling in the region where annual rainfall lies between 100" - 300" possess quite a low density (viz. 60 - 75 lb/cft). Such soils would not be able to support high earth dams, but it may be only incidentally that the soil having such a low density has been found at Cheruvathy site having rains between 100" - 300". Swift Dam No.1 (U.S.A.) was actually constructed in a wet region where average annual rainfall is 125". In this context, founding earth dams in wet regions cannot be wholly rejected. If Cheruvathy Dam was met with a low density overburden, it may be a chance, because it was not true in case of Swift Dam No.1, both falling in Rao's classification of 100" - 300" rainfall. Compaction of low density soils will however, have to be done with care and choice. Light Plain Rollers (1 Ton) may be of use for such soils.

Soils in the second or third classification of Rao have very close condition for compaction purposes, though they are being used in different zones of impervious core or shell material alike. Sheep foot rollers, elephant foot rollers, rubber tyred rollers, and tamping rollers[8], grid rollers[8] or segmental rollers can be employed on both zones without discrimination. Their size will have to be decided by their actual efficiency to be noted at site. This can be better decided by running test sections and using different rollers with different moisture content and number of passes.

6.2.3. Remarks

In cohesive soils, following types of rollers can be decided for use after running test sections.

SHEEP FOOT ROLLERS

Their size, (total weight of roller and stress per unit area etc) can be decided upon at site by actual observations and their proficiency on test sections. A combination of sheep foot roller with others are sometimes more economical than its being used singly. At Sasumua Dam (Kenya) a peculiar tropical residual soil needed compaction in 6" layers first with a 20 tons sheep foot roller, then with a 30 tons rubber tyred roller and finally with a 45 tons rubber tyred roller. This avoided likely stratification of the fill. This type of roller (sheep foot) helps in breaking clods and kneading, apart from applying heavy load required for a homogeneous compaction. At Hirakud Dam, sheep foot rollers with 9" layers rolling worked well at placement moisture of 1 to 3 % less than O.M.C. Sheep foot rollers are commonly available in the market in 6-14 sq. in. foot area and 250 - 600 p.s.i. unit (U.S.B.R. Specifications). The unit foot pressure of these rollers have been manufactured in United States upto 1100 p.s.i. for specific purposes.

ELEPHANT FOOT ROLLERS

Rollers of this class have flatter feet to increase per unit foot area from 6 - 14 sq. in. to reasonably desired sizes which reduces per unit foot pressure from 250 - 600 p.s.i. to 144 sq. in. or even less. Some soil having low shear strength (viz. silty clay) cannot be compacted to required proctors density by sheep foot rollers. Elephant foot rollers, help compaction of such soil to a certain extent. An exact problem of the kind was encountered at Ukai Project (Gujarat) where the soil analysis showed to contain 10 - 40 % sand, 50 - 60 % silt and 5-10 % clay. Though this gave a satisfactory proctor's density in the laboratory, use of sheep foot rollers did not give more than 88 % compaction. The soil cheesed off because of higher %age of silt content before being further compacted. Use of elephant foot rollers and rubber tyred rollers however increased compaction from 88 % to 98 %.

RUBBER TYRED ROLLERS

In above cases, where even the elephant foot rollers fail to give more than 80 - 88 % compaction, heavy rubber tyred rollers are more useful having 80 - 100 p.s.i. inflation pressure with wheel loads from 10,000 - 23,000 lbs (U.S.B.R. Specifications). Ukai Project is using it for compaction of its clayey silt material of above composition.

TAMPING ROLLERS

In D.V.C. Dams, with their average proportion shown in Table No.4.1, compaction was done by tamping rollers, having staggered ball foot or knobs, uniformly spaced to provide at least one knob for each sq. ft of area. The unit pressure was about 400 p.s.i. and it was found that 8 to 12 passes gave satisfactory results. For so much of contact pressures (400 p.s.i.) sheep foot rollers or even heavy rubber tyred rollers also would have successfully compacted the soil. It appears choice of tamping rollers was arbitrary and not based on study of economics, having made on test sections.

PNEUMATIC ROLLERS

In earth dams construction there are occasions where rollers cannot be employed for lack of space or for reasons of workability. One such occasion arises at the junction of earth dam with masonry structure. Pneumatic rollers are usually most suited to such sites and compaction can be completed in smaller thickness of 6". At the sides of cut off trench also, pneumatic rollers will be helpful in compacting soil. Over foundation of hard rock soil should preferably be compacted in 6" layers first with hand ramming and then employ pneumatic rollers until it becomes workable with sheep foot or other rollers or rubber tyred rollers.

Above are only basic economics of some of the important types of rollers employed on cohesive soils for compaction. Their final selection lies with the Engineer Incharge after seeing their proficiency on test sections.

For compacting sand and gravel fills, following resources are generally employed:-

(a) Plain rollers
(b) Vibratory rubber tyred rollers
(c) Rubber tyred rollers
(d) Internal deep vibrators (Vibro-floatation)
(e) Explosive blasting
(f) Air Injection (Blowing compressed air)

PLAIN ROLLERS VERSUS RUBBER TYRED ROLLERS AND VIBRATING RUBBER TYRED ROLLERS

Smooth rollers work well on thin layers of sand or gravel (say 6"). But since the initial layers of sand or gravel fill in loose state have low shearing strength, better results can be expected out of rubber tyred rollers only, due to flexibility of its tyres which reduces the peaks of the surface loading to the amount of inflation pressure. If the gravel or sand is mixed with silt, say a case of silty gravel or silty sand fill, again vibrating tyred rollers do not work well because of mortification of vibration by the rubber tyres. Exactly similar, a case was observed when efforts were made to compact a test section in

France consisting of silty gravel in 12" thick layers by a 16 - 27 tons vibrating rubber tyred rollers. On the contrary a 6 tons sheep foot rollers (a comparatively light weight roller) with specific pressure of 340 p.s.i. gave satisfactory results. Apparently though the former was very heavy roller, it could not give required contact pressure to compact the fill. It might be possible that without vibration, simple rubber tyred rollers could have given a better account. If soils are placed on embankment wet of O.M.C. due to adverse climatic condition (wet weather) or for reasons of speed and progress, heavy rubber tyred rollers are to be preferred for compaction of sand or gravel fill. It also gives satisfactory results on low-shear silty soil etc. with light weight rubber tyred roller.

In case of 515' high Swift No.1 Dam (U.S.A.) constructed of selected and random alluvial granular material in the core and in the shell respectively, the compaction was successfully done in adverse climatic conditions (average annual rainfall 120"), by a 50 tons rubber tyred roller in 15" lift in the core and 24" in the shell. The impervious core and the U/S and D/S random granular shell sections were compacted with 4 passes whereas sand and gravel transition zones required 2 passes for obtaining desired compaction (95 % of laboratory value). It is obvious, therefore, that the use of heavy rubber tyred rollers allows the borrow materials to be placed in relatively thick lifts and thereby results in economy. Another example of test sections having been adopted for deciding size and type of rollers can be cited of that of Rosshaupten Dam (Germany). At construction site, compaction test of varying duration were undertaken with the moraine gravel (containing 50 - 70 % of fine material 4) subsequently employed in construction, in layers of 10", 20" and 40" and different moisture content, using heavy vibrators ranging from 4 to 8 tons. These tests revealed that almost irrespective of the fill height, and with a moisture content of 6.8 % resulting in a sample to saturation after compaction, maximum compactness could be obtained. Even at a fill height of 40" uniform compactness was achieved and comparative tests proved that the 8 tons vibrator (imparting strong horizontal vibration of 2000 rpm to the soil mass) was the most effective appliance for the job.[60] It increased the angle of friction from 37.0° for uncompacted soil to 43° for compacted fill. The high cost of compaction was thus compensated by savings resulting from a reduced volume of the dam.

INTERNAL DEEP VIBRATION (VIBRO FLOTATION), EXPLOSIVE BLASTING AND AIR JET GROUTING METHODS

These compaction methods are usually chosen, when large sand mass deposit is to be compacted for fear of liquefaction or for increasing shear strength of the foundation, particularly consisting of fine sand. Different firms have got their patent equipments for achieving such compaction, but all have the same principle in common, viz. the sand mass, is allowed to settle by creating artificial vibration by any of the above means and forced to compact by reducing voids. Again the choice for one or the other device will depend upon studying the relative economics at construction site in a small scale. In case of Sadd El Aali (High Aswan Dam on the Nile)(Egypt) use of heavy internal vibrators were found to be more economical than the explosive blasting, in compacting dune sand fill placed under water. It increased penetration resistance from 60 kg/cm^2 of the uncompacted fill to over 190 kg/cm^2 recorded at 16 m depth in the sand fill.[61] However, this method of vibroflotation has not been tried elsewhere on so much large scale as at High Aswan Dam and its effectiveness at other dams still remains to be testified verified in the context of local conditions pertaining to Indian Dam sites. Use of 'Air Injection' (blowing compressed air) for compacting sand masses in under water state has been successfully recently tested at U.S.S.R. with Maslov's method in conformity with the suggestion of Engineer Grigoryev and others.(78) Its use also on larger scale on High Earth Dams yet remains to be studied before adopting it for dam compaction with confidence in our country. Still there can be no doubt that these methods have a good future where earth dams are required to be built on pervious foundation.

SPECIALLY EQUIPPED 8 TON ROLLER

For compacting transitory types of soil between gravel and sand on the one hand, the impervious clay on the other, both sheep foot rollers and rubber tyred rollers are in use for compacting at approximately optimum moisture content. However, for working on wet of O.M.C. Tricker(80) has reported a new type of 8 tons smooth rollers employed on Rosshaupten Dam (Germany) which worked very effectively there. This 8 tons smooth roller had shoes attached on its drums, interlinked with articulations, so that it touched the soil surface lifted from it vertically. This avoided tearing off the already compacted fill. The boulder clay containing more than 30 % sand and gravel was laid in 8" thick layers and compacted by 8 passes of this roller to dry density of 2.30 T/m^3 at 7 % moisture content.

For a similar soil at Serre Poncon Dam (France) the core consisting of silty moraine with nearly 65 % sand and gravel, adequate compaction at 2 % above O.M.C. was obtained with sheep foot rollers followed by rubber tyred rollers. This type of soil being less sensitive to moisture changes, is able to compact in thicker layers (say 12") than the above type (cohesive). This also ensures higher density of compacted fill. In India too, time has now come to evolve and employ only compaction machines for most suited to local conditions without going for standard and conventional machines.

6.8.8. [illegible]

Shear strength of soil fill is given by $S = C + \bar{\sigma} \tan \phi = C + (\sigma - u) \tan \phi$. In case of cohesive soil, compaction improves friction angle " ϕ " and total stress ' σ ' of the soil by increasing density of the soil. At the same time construction pore pressure 'u' plays an adverse role. With greater moisture content in the soil construction pore pressure increases with increased load of rollers and this results in reducing effective stress. It is also a fact that the water content to a certain limit, helps compaction of soil itself working as a lubricating agent between soil grains. Hence only test section can be a correct guide to decide 'what is the optimum workable water content' (instead of trying to obtain compaction at O.M.C. at site) at which a heavy roller (may be specially designed one) would give desired compaction at fewer passes. Test section also decides weight and type of rollers. In case of cohesionless soils, compaction increases friction angle " ϕ ", substantially, at 70 % relative density. The desired maximum compaction is obtained when the sand is saturated. Hence, compaction of sand or gravel is always assured by employment of heavy rollers accompanied by profuse watering. Since sand or gravel is free draining, pore pressure does not develop and hence increase in " ϕ " directly increases effective stress σ, a difference from cohesive soil.

6.3. SUMMARY

For different soils, following compaction machines and methods will be useful although their size/proficiency/workable water content for compaction/number of passes for required compaction etc. will have to be studied at site on test sections before finally adopting one or the other for field use.

(1) ROCKFILL

(a) HARD ROCK

Dumping in high lifts and sluicing with heavy water jets is quite a fast process and so progress is accelerated. Compaction by rubber tyred rollers in 2' - 3' thickness gives slightly better results but progress is sluggish. Sometimes rolling and vibrating may also be of use if compaction is preferred in thin layers.

(b) SOFT ROCK

Compaction by heavy rubber tyred roller, sometimes combined with sheep foot rollers, vibration and (or) sluicing can be tried at site and individual compaction preferred on its merits only.

(2) GRAVELLY AND SANDY SOILS

Different machines for compaction are heavy rubber tyred rollers, vibrating rollers, internal vibrators. Sometimes use of sheep foot or elephant foot rollers will be of good avail when sand or gravel is with silty clay and their selection can be made as per site condition based on economics.

(3) COHESIVE SOIL

Better compaction better by heavy sheep foot rollers in relatively thin layers (6") or by heavy rubber tyred rollers in 12" layers will give satisfactory results. Control of construction pore pressure will have to be ensured in either case.

It is evident that controlling compaction effort primarily depends upon correct choice and size of compaction machines and compaction methods. A heavier machine than what is required for a particular soil will simply disturb the compacted soil, while a too light machine will start "walking" on it without giving tangible results or it may need extra number of passes resulting in extra cost. In a dense fine sand deposit, sometimes internal deep vibrator may prove to be more economical than taking recourse to explosive blasting or "air injection". Similarly, compaction of sand or gravel in thin layers (6"-12") by use of vibrators will be more effective when accompanied by profuse watering than its being compacted dry, while compaction of cohesive soil is better achieved with sheep foot rollers near O.M.C. and use of still heavier machines gives quicker compaction.

CHAPTER - 8

EARTH DAMS FAILURE DUE TO CONSTRUCTIONAL LAPSES AND THEIR REMEDIAL MEASURES

8.1. CONSTRUCTION FAILURE DEFINED

An earth dam can fail in many ways, important of them being by

(i) Overtopping

(ii) Piping through or under the dam

(iii) Excessive settlement causing severe cracks, and

(iv) Failure of filter causing drainage troubles

Failure of earth dam by overtopping may result due to insufficient spillway capacity, or due to inadequate provision in free board. Both of these reasons pertain to failure of Dam Designs and a Construction Engineer cannot help it. In other cases of earth dam failure, provided sufficient provisions and counter-measures have been taken in designs and still, if structure fails, it is indicative of mistakes somewhere in construction. Many cases of construction failures have come to light and a few of them are discussed below to illustrate how certain lapses while construction resulted in partial or total failure of earth dams, which would have been easily thwarted with a little more understanding of the subject and a vigilant control over the work.

8.2. EXAMPLES OF CONSTRUCTION FAILURES AND REMEDIAL MEASURES

1. NURAURA DAM (M.P.)

Above dam failed while construction in the year 1953. After closure of the river in June 1953, water started rising in the reservoir and supplemented by heavy rains on July 8, 1953, it reached a height of 30', D/S water level being 37' below that of U/S. Water started coming out, adjacent to the masonry sluices on the right, accompanied by a third peak close to the U/S toe near entrance to the sluice. Within 3 hours of leak, developed at D/S face it enlarged to a breach of 100 ft and the dam failed. At the beginning of the breach, water was observed coming out at the D/S rock toe level and the filter at the back of the rock was first blown off before the breach occurred.

At after failure study revealed that the filter and rock toe adjacent to the sluice, were omitted in a certain length during construction. This did not allow seepage pressure to dissipate at the D/S toe causing subsequent soil erosion in the passage of water through dam. Due to considerable soil movement, earth section above it, subsided followed by a slide of earthen section laid on the hill side slope. This great force during slide must have deflected the masonry side wall on the river side, resulting in the collapse of the masonry arch first. This led 37 ft water head behind gates to force entry through the collapsed sluice-way causing great vibration and additional settlement in the adjacent earthwork. Piping increased through the loosened soil and this resulted ultimately in 100' wide breach. It was later confirmed from tests of soil samples that the compaction between 91.2 % - 98.6 % was achieved on the embankment. Evidently, compaction was satisfactory and this might not have contributed to failure. A construction lapse of omitting filter in a certain reach resulted in the great mishap. The dam has since been rebuilt with adequate provision of filter at the junction.

8. PALAKMATI DAM (MADHYA PRADESH)

Palakmati Dam 145' high and 8200 ft long, was built on black cotton soil overlying clayey soil mixed with pebbles, both of which, when saturated acquire very much lower shearing strength.

During heavy rainfall, after 15 years of construction, a slide occurred on the upstream slope of dam resulting from settlement of foundation in the year 1963 (June). Evidently failure had occurred due because no adequate precaution had been taken to improve drainage below the earth dams i.e. in foundation.

Remedial measure lay in [illegible] flattening the upstream slope from 2:1 to 3:1 and providing a rock toe on the U/S.

5. MOORUM NALA TANK BUND (MADHYA PRADESH)

Moorum Nala Tank Bund, 62' high and 6250' long on first filling in September 1963 developed severe loss of water by seepage appearing on the D/S slope. To restrict excessive seepage through body of dam, it was decided not to allow water to rise above 10' below full tank level and a waste weir was proposed in the earthen bund at this level. The bund, however, breached while the weir was still being constructed, because the seepage pressure had developed to dangerous proportions by that time. The weir was reconstructed in the flank and it worked well till 1964. In that year, again the water level rose to 40' height, due to inadequate spilling capacity of the weir and excessive seepage led to D/S bank slide of the earthen bund. Two causes appear to have contributed to this failure - first the insufficient control of embankment material and its compaction and the second the absence of filter at the D/S, However, a corrective measure to reduce seepage lay in providing a clay blanket on the U/S side and a rock toe on the D/S side.

6. ARUAR DAM (RAJASTHAN)

41' high Aruar Dam (Rolled fill and zoned), constructed in the year 1956-58 subsequently failed twice once in the year 1958 and again in 1957. Reasons were unsatisfactory compaction of the earth work and inadequate cutoff below the dam. Former resulted in excessive settlement and seepage through dam, while latter was responsible for high uplift pressure for underground seepage.

5. <u>ASHTI DAM (MAHARASHTRA)</u>

58' High Ashti Dam (Rolled Fill) constructed during 1878-83, resulted in failure sometimes between 1883-93. The dam was built on weak foundation material which might have got saturated in course of time and acquired low shear strength, resulting in plastic failure and slip on D/S face. Lack of proper drainage facilities in the foundation contributed to this failure.

6. <u>GUDDAH DAM (RAJASTHAN)</u>

93' high rolled fill and zoned Guddah Dam was constructed in the year 1956. Trouble started at the junction of right sluice training wall in 1956 itself, when the reservoir was first filled and the dam failed because of inadequate compaction resulting in settlement and seepage through the Dam.

7. <u>PANSHET DAM (MAHARASHTRA)</u>

This dam was built on River ~~Mutha~~ Ambi, and was aimed at to supplement the water in reservoir, behind Khadakwasla Dam. (over Riv Mutha) However, the construction was yet not complete and monsoon floods arrived. The gates which were still under erection, remained half way in the outlet sluices built in the dam. The approach bridge being not yet ready, gates could not be operated to make it full open. Sudden monsoon floods rose to dangerous heights, resulting in enormous velocity below partly open gates for which it might not have been designed. Construction of flow near the shoe of the partly open gate caused too much vibration and cavitation to damage masonry outlets and resulted in its ultimate collapse, subsidence and a wide breach in the adjacent earth sections. Evidently, locating outlet sluice in the dam poor time schedule and keeping the gates in partly open position resulted in great mishap, which ultimately breached Khadakwasla Dam (masonry) also, about 10 miles downstream. The latter was overtopped, but it actually failed while the water started receding. A later study on photo-elastic model of Khadakwasla Dam revealed that the masonry section failed due to differential settlement caused by concentration of flow at a point where the foundation was at a still higher elevation than the adjacent portions, conforming to geological profile of River at Dam site. In case of Panshet Dam, locating construction sluice with the body of the earth dam was itself a mistake and in any case, the operation of gates should have been well ensured, before arrival of monsoon floods. After failure of the dam, restoration works were taken up during 1963-67 and the outlet sluice has been replaced by waste weir on the flanks.

C. NANAK SAGAR DAM (U.P.)

64' high rolled fill and zoned, Nanak Sagar Dam over River Deoha constructed in the year 1962 in Nainital District of U.P. recently failed in the first week of September 1967. Properties of materials, used for construction were as shown below:-

	Core (CL and ML-CL)	Casing (SM-SC)
Specific Gravity	2.68	2.68
Maximum dry density	100 lb/cft	107 lb/cft
Optimum moisture content	18 %	15 %
Coefficient of permeability	2 ft/year	20 ft/year

The dam was constructed at 1 % wet of optimum moisture content, and density control was obtained by varying the compactive effort to suit the moisture conditions. Dry density figures attained in the fill were generally on the higher side of the Proctor's Optimum Value, the standard deviation being 7 lb/cft over the average figure of 103 lb/cft. Though compaction achieved was commendable, high construction pore pressures were noted (because of wet construction), during construction of the 84' high dam in one season and record of piezometer readings revealed low dissipation of pore pressure in the core subsequently. Right after the construction of the dam in the year 1948, some leaks appeared at the D/S toe and it increased with increase in head of the reservoir water level. Foundation of the dam consisted of the 3 m silty clay (ML-CL) underlain by 9 m poorly graded sand (SP) and then again silty clay (ML-CL). This soil stratification in foundation resulted in saturating the silty clay (which gradually acquired lower shear strength) followed by increase in pressure below it because of a pervious layer sandwitched between two layers of silty clay (ML-CL). Naturally, with increase in water head in the reservoir, this hydro-static pressure also increased when uplift below top silty clay layer exceeded the column weight of the latter, it might have amounted to bursting of the strata appearing in sand boils first that led to failure by piping. It is believed construction of adequate pressure relief wells would have saved the structure from failure. Another example of Sarda Sagar shows how the structure has survived so long because of providing adequate pressure relief wells, which is also founded on stratified foundations.

C. SARDA SAGAR DAM (U.P.)

This 16 miles long dam with maximum height of 57' is founded on Alluvium, consisting 3' to 10' of relatively impervious layer of silt or clayey silt overlying 100' - 120' of pervious sand

and gravel and then again a clay bed. The geology presented a serious problem of sand boil after construction of dam in two stages in 1957 and 1960. After observing first trouble in 1957, remedial measures taken to reduce seepage pressure consisted in U/S impervious blanket and pressure relief wells on the D/S. The dam worked satisfactorily at low reservoir levels after that but in the year 1963, five years after initial filling of reservoir when the water rose to 60' height, the dam started sloughing on D/S in a length of about 1500' and serious boiling conditions started downstream of well line in a reach of about 6800 ft. Site exploration revealed following reasons for failure at two points.

<u>At M 7-3-0 to M 7-6-0</u>

A bore hole on the D/S face revealed silty material laid over filter toe, which itself being impervious one caused stratification on account of which phreatic line was raised and seepage started emerging on the D/S slope. As remedial measure, silty clay material was removed from over the filter and replaced back by sand. This worked well since effective drainage was ensured.

<u>At M 7-8-0 to M 7-9-0</u>

In this reach, serious boiling was observed on October 21, 1963. An investigation established that present R.C.C. slab of the drain bed in this reach had subsided by about 3 ft. A large sand boil also appeared at a distance of 15 ft from the outer edge of the longitudinal drain. Three days later the inner edge collapsed in two new reaches and the following day the drain bed subsided by about 6 ft in a length of 15 ft. A study by installation of a few observation pipes at D/S revealed a pressure of 7' water head, having developed below 3' - 6' impervious clay. The latter got consequently lifted up resulting in numerous boils on the D/S. To get rid of situation further pressure relief wells in two rows and seepage berms were provided.

Besides, above there are numerous other examples where earth dams have failed for one reason or the other, for inadequate provisions while construction. Such failures can be forestalled only by studying special conditions pertaining to site and taking measures against them besides being careful in execution of the works as per terms and specification. Construction problems of some of the Indian Earth Dams as encountered during construction are taken up for discussion below.

6.3. ENCOUNTERED CONSTRUCTION PROBLEMS

1. GANGAPUR DAM (MAHARASHTRA)

This dam, maximum height 123' and 12,300 ft long, had to be built on black cotton soil. For this, bearing strength of the clay under saturated conditions was determined and the section was accordingly adopted for the dam. Precautions finally taken comprised of stripping the black fissured clay to a depth of 8 ft, digging cutoff and backfilling with compacted clay and providing longitudinal and cross drains inside the dam and an outfall drain 20' outside. The internal drains were filled with river gravel, and the outfall drains with graded filter. In the river bed, foundation comprised of 10' maximum depth of gravel on good hard rock. This did not pose much of problem and the foundation of dam was laid by stripping top 8' gravel. Internal drains were, however, provided in this case also leading to an outfall drain pipe.

2. D.V.C. DAMS

First two dams built by D.V.C. were Panchet Hill, on River Damodar (West Bengal) and Maithon on River Barakar (Bihar), the river portions being completely closed by earth dam sections. The river bed consisted of 70'-80' deep sand over hard rock. Various seepage control measures including sheet piles and U/S blanket were considered for earth dam section in river bed and ultimately both of impervious blanket and sheet piles were provided. The dams are working satisfactorily.

3. KOTA BARRAGE (RAJASTHAN)

Earthen section, 122' high has been built across Chambal (Rajasthan), the river bed consisting of 40'-60' thick boulders of different sizes and sand. Open cutoff was very much expensive hence other alternatives were considered. Sheet piles driving was tried in a certain reach of the section but it did not become successful. Cement clay grouting had been adopted as an alternative to sheet piles, it worked better than the former but seepage loss is still enormous after construction that can be seen at the site.

4. HIRAKUD DAM (ORISSA)

At 100' high Hirakud Dam, not much of problems arose during construction of earthen section. The dam alignment crossed at one place obliquely in the river bed, formed of deep rock channel after long erosion. The earth was laid and compacted here after pumping out water and taking usual precautions of removing all loose rocks. The dam section was raised sufficiently high on the U/S to prevent the river passing over the earth filling section during the monsoon. The downstream, however, got submerged under influence of backwater, earth section having been not raised as high as on U/S.

C. <u>OBRA DAM (U.P.)</u>

This dam still under construction is faced with unique geological problems. River bed comprises of 70' deep coarse sand overlying limestone, often met with large crevices. Cutoff trench excavation and backfilling with impervious clay required enormous dewatering and its idea was abandoned in view of a novel technique developed by M/S Rodio Hazart and Foundation Engineering Co. They adopt Rodio Marconi Method for trenching 1 m wide, side sand walls remaining in position due to use of heavier than water Bentonite solution, and after reaching required depth of foundation, the trench is back filled with tremie concrete (1:3:6) having 7" slump (approx) which sets in position after replacing bentonite solution, being much lighter than concrete. They are cast in convenient panels say 40' - 60' wide and give very rapid progress. The processes of trenching and concreting are adopted in alternate panels to allow time for concrete to set. In foreign countries, they have adopted one row of concrete diaphragm, as concrete cut off and it has worked there satisfactorily but this being first experience in India for Earth Dams, double row concrete diaphragms have been adopted at Obra, 6 m apart and the intermediate sand having been treated with chemical grouting. Rock below the diaphragm being met with large crevices, have been adequately treated by cement grouting. Liquefaction of Obra sand under earthquake intensity of 8, so far observed in the region, has been ascertained by tests to be a distant probability and hence it warranted no further treatment, against this.

C. UKAI DAM (GUJARAT)

Project authorities had a great problem as to how to use silty clay material (silt 80 - 60 %) available in abundance at construction site. This clay could not be compacted with known implements by more than 90 % at site, though its proctors density in Laboratory was ascertained to be satisfactory. An outright rejection of the soil, available in large quantities for use in the dam would have been quite uneconomical. They got over the problem by deciding to use this material on the D/S portion by adopting a rather unorthodox type of zoning of the earth dam, with a more inclined zone of the impervious soil. This material was not considered suitable for being used in the U/S zone above low water line, because the stability of the material during sudden drawdown was doubted. Another problem arose out of constructing a positive cutoff in the river bed, where water stood 20'-40' above bed level, the dam being on U/S of Kakrapara Weir. This threatened large scale seepage while constructing positive cutoff. This has been encountered by constructing concrete diaphragms by Rodio Marconi Method, one on either side of the cutoff trench, and then proceeding with excavating pervious material in between and backfilling with compacted impervious material.

13. TENUGHAT DAM (BIHAR)

(a) 162' high Tenughat Dam has some of its problems similar to that at Obra Dam and in that context similar treatments have been adopted in river bed section, river Damodar bed having 60 ft deep coarse sand, overlying fissured and jointed Gneiss rocks. Two sets of concrete diaphragm 1 m wide and 6 m apart have been adopted and sealed to foundation rock bed. Intermediate sand between the two diaphragms and the fissured rock beds are to be grouted, after this monsoon. (June '67 to Oct '67)

(b) Full height of 142' high earth section in the river bed width (1000 ft long) is to be completed in 8 working months starting from November 1969. In context that the construction will be quite rapid, all precautions are being taken to keep the construction pore pressure to minimum by observing piezometer and providing adequate drainage arrangements at intermediate levels. In the region where, Tenughat Dam is being built, unit weight of impervious and semi-pervious materials are quite close viz 122 and 119 lb/cft and permeability vary between 3 to 10 ft/year. To improve drainage in shell material of the dam, even on the flanks, both on U/S and D/S of the core was a great concern of Designers. The same has been overcome by providing intermediate drainage blanket on the U/S face and inclined chimney on the D/S (vide Fig.7.8). The U/S blanket will improve drainage especially during sudden drawdown conditions, while D/S chimney will improve drainage during steady seepage state, at different reservoir water levels.

(c) Coarse sand in River bed extends to 60' depth and it has been established after simulating earthquake conditions, by explosive blasts that it would not liquefy under earthquake of intensity 8, for which the dam is designed. This ensures safety of the foundation against liquefaction apart from eliminating infructuous cost which might have been incurred to compact the river bed sand. Hence, no further treatment of the river sand is being proposed.

(d) In some reaches, strata below cutoff bottom levels extending deep underground are likely to cause enormous loss of water due to seepage if no treatment is provided. Hence such regions are being probed by in-situ permeability tests and they will be treated with clay grouting subsequently.

(e) At dam site quarries are located quite afar. Hence insistance on using graded filter would have resulted in enormous expenditure. An alternative to it lay in using river sand which as well satisfied ~~further~~ filter criteria. The same has since been adopted for use.

Several other construction problems may arise for a particular project which can be overcome only after investigating the facts, analysing and taking decisions with a good understanding of subject.

CHAPTER - G

SPECIAL CONTROL MEASURES

G.1. STUDY OF EARTHQUAKES AND LIQUEFACTION TESTS

G.1.1. EFFECT OF EARTHQUAKES

Several earth dams constructed before 1920 A.D. were not actually designed for earthquake consideration, still they have withstood small or big earthquakes without much of damage. A few of them have of course failed, but they were relatively small dams not exceeding 100 ft in height. An obvious inference may be that these dams which did not fail, were better built and compacted than the other ones, but there is no evidence to support either way. No systematic study of such failures were taken up prior to 1950. Ambraseys(61) was the first to collect such informations in the year 1960-62. Since then, others have also worked on failure of earth dams and have published informations on some of the important dams. Some cases are given here in Table No.G.1 (attached). Most of the earth dams given in the above Table were constructed earlier than 1920, when knowledge of soil mechanics and earth dams construction was not very advance. All the same, informations gathered will be a valuable guide for upcoming new projects, where every efforts are made to take up construction of earth dams with as correct predictions for earthquakes as possible.

Table No.G.1 reveals that earthquake affecting earth dams may cause one of the following types of failures:-

(i) Compaction of the dam and/or foundation and consequent slumping and possible overtopping by water.

(ii) Sliding of dam on its base

CHAPTER - 6

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FAILURE OF EARTH DAMS DUE TO EARTHQUAKES

TABLE NO.1

S.No.	Dam & Name of country where built	Year of Construct-ion	Type of Construction and brief history about dams	Characteristics of Dams				Year and Regional name of earthquakes	Maximum intensity & at site intensity of earthquake on (MM), (RF), (JMA) Scales (see Fig.7 P.421 for scale definition)	Physical Damage to dam due to earth-quake	REMARKS
				Ht ft	Length ft	Crest ft	U/S Slope				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	Langley (U.S.A.)	-	Earth Dam situated about 100 miles from epicentre of earth quake - overburden in the neighbourhood was loose soil	-	-	-	-	Aug.31,1886 (Charleston Earthquake)	VIII (R.F)	Transverse cracks enlarged to breaches (shear failure)	Loose soil in neigh-bourhood of dam might be one of the reasons for failure
2	Greggs (U.S.A.)	-	Earth Dam (= 10 miles from epicentre	-	200	-	-	-do-	IX (R.F.)	Longitudinal cracks slides on both sides of dam, settlement of dam by a few foot causing (failure by overtopping)	Short distance from epicentre has chances of settlement and failure by over topping
3	Ikena Zeno (Japan)	Built after 1920	Earth Dam - Concrete paving on U/S face	50	300	10	2	March 7, 1927 (Tango Earth-quake)	X (R.F.)	Settlement of dam into soft foundat-ion bulging at U/S toe - Breach at left abutment	Foundation failure due to inadequate compaction
4	Ida (Japan)	-	Earth Dam	30	-	-	-	Nov.26, 1930 (Idu Earthquake)	V (J.M.A)	Failure suspected at outlet pipe crossing at its deepest foundation (foundation failure)	Avoid pipe crossing in the body of earth dam. Junctions are notably weak points Similar factors at Panshet Dam (India) also occurred but the vibration caused was not due to E/Q but floods
5	Usonte (Mexico)	1937	Semi-hydraulic dam founded on stiff blue tertiary shale U/S concrete pavement	100	4200	20	3:1	Dec.8,1939 (Kagi Earthquake)	V (J.M.A)	U/S & D/S slopes slide. U/S slide continued long after the Earth quake	This is a case of high dam. Failure appears to have occurred due to insufficient compaction.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
6	Hunokawa (Japan)	-	Earth Dam			14	0.1	May 1, 1939 (Oga Earthquake)	V (J.M.A)	Failed with first shock, hence type of failure could not be established	
7	OY (Japan)	1939	Earth Dam - founded on dense gravel-bed overlying granite into which a drain was cut to carry an outlet conduit		610	88	3.0:1	(a) Jan. 13, 1945 (Mikawa Earth quake) (b) Dec. 21, 1946 Nankai Earth quake)	V (JMA) III (JMA) at site V (JMA) IV-V at site 170 miles away from epicentre	No damage	Same comments as for S.No.6
8	[illegible] (Japan)	1949	Earth Dam - built on thick alluvial deposit 160 miles away from the Nankai epicentre	68	940	30	3:1	Dec.21, 1946 (Nankai Earth quake)	V (JMA) (IV JMA at site)	Ground near dam fissured and major leakage from right abutment (½ million gals a day) - outlet pipe in foundation fractured. Crest settled by 6 inches	More cases are warranted at abutments of Earth Dam or at other junctions. Materials at abutments should preferably be high shear resistant
9	Choato [?] (Japan)	1920	Earth dam built on residual clay deposit overlying conglomerates Shoulders (casing) of dam built with conglomerates and silty clay. Central core of silty clay epicentral distance 170 miles	70	618	26	3.0:1	-do-	V (JMA)	Structure slumped cracks on D/S slope at 1/3 of ht. After Earthquake repaired with clay grouting	Construction material silty clay used for central core might not be working as effective impervious core & seepage might have increased with earthquake shocks causing their failure
10	Hichiiron [?] (Japan)	1709	Earth Dam in the Hiro shima. Prefecture founded on sound rock. No stability trouble before earthquake (Epicentral distance 210 miles)	63	280	30	3.0:1	-do-	-do-	Dam cracked to heavy leakage though old dam, & though situated afar from epicentre	Old dam, but the fact that the dam had not developed any trouble before E/Q shows a good workmanship, with that it must have been built as far back as 1700

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
11	Kisenzaki (Japan)	1949	Earth dam in Hiroshima prefecture founded on solid rock, epicentre 200 miles away	60	200	18	2.7001	Dec.21,1946 (Nankai Earth quake) (IV-JMA in the region)	V (JMA)	Only danger at outlet pipe	Same remarks as for S.No.6
12	Sakima (Japan)	1539	Earth dam founded on soft rock and has been subjected to Eqn E/Q many a times. Epicentre 25 miles	118	920	80	-	VII	V (JMA) XV (JMA) at site	Body of the dam fractured but did not collapse	Old dam. Good workmanship in construction
13	San Andreas (U.S.A)	1870	Earth Dam - Fault enter reservoir area and cross the rocky knoll in between two embankments Foundation material immediately below the dam consists of gravelly stratum mixed with sand and clay	93	620	20	3.001	April 18, 1906 (San Francisco E.Q.)	X (R.F.)	The rocky knoll was shattered and top of dam shattered by 7'. Both longitudinal and transverse cracks developed. After 1906 E.Q. dam was repaired by E V. surfacing over the knoll	Dam alignment should preferably be avoided of crossing fault underneath. Longitudinal settlement may be due to settlement of foundation.
14	Ono (Japan)	1913	Earth Dam - Embankment and core materials were well compacted and seepage was prevented by two puddled clay core walls. Right abutment on paleozoic rocks but rest of the dam rests on the conglomerates	126	1000	26	2.001	September 1, 1923 (Kwanto E.Q.)	IX (R.F.)	Dam fractured in many places. Fissures 100 - 100' long formed and crest settled by 1 ft.	
15	Sheffield (U.S.A)	1917	Earth dam founded on sand stone dipping slightly in D/S direction Dam is made of pit run material from transverse excavation.	25	720	20	2.001	June 29, 1925 (Santa Barbara E.Q.)	IX (R.F.)	D/S portion of central part of the dam moved out but the D/S face with grass sodding remained intact.	Different experts have opined on this and attributed the reasons to liquefaction, unequal settlement, fault crossing beneath the dam etc.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
16	Chatsworth Dam (U.S.A)	1918	Earth Dam No account of construction available	66	2700	31	2.0:1	August 1930 (Santa Monica Bay E.Q.)	VIII (MM) VI (MM) at dam site	Seepage through dam increased, U/S slope slide and longitudinal cracks on both U/S & D/S slopes. Thickness of varied from 1/8" to 3", 6'-12' deep into the dam	
17	South Haiwee Dam (U.S.A.)	1913	Semi hydraulic dam founded on silt stone 90 miles away from epicentre	91	1950	20	2.0:1	July 21, 1952 (Arvin-Tehachapi E.Q.)	X (MM) VII (MM) at site	Cracks on large scale however the section did not slide on slope settlement of crest by 1"	Example of damage due to heavy intensity earthquake (n)
18	Hebgen Dam (U.S.A)	1915	High Dam - No account of construction available	87	720	20	3:1	August 17, 1959	X (MM) VII (MM) at site	The whole crustal formation of the lake underwent severe tilting and shore lines emerged by 8' in the smaller part of lake. On the north west part shore line submerged by several feet. This tilting caused waves of 3' height resulting in failure of dam by overtopping. The fill on both sides of the core cracked and slumped	Example of damage due to heavy intensity earthquake (

(iii) Slipping of the slopes resulting in longitudinal cracks near the top.

(iv) Separation of the dam from its abutments or shearing of a dam across its section if the fault crossed it.

(v) Failure by resonance.

Having known the effects of earthquakes on earth dams we will study in subsequent para, how to take steps to ~~counteract~~ counteract their effect while constructing ~~them~~ dams.

6.1.2. STUDY OF SEISMIC REGION AND COUNTER MEASURES AGAINST EARTHQUAKES

(A) To provide adequate safeguards against earthquake failures, the region where the dam structure has to be built should be thoroughly probed for seismic informations as explained below-

DEFINITION OF EARTHQUAKE

Earthquake is caused by the release of strain energy stored in earth crust due to various causes. The energy is dissipated through transmission of compression wave (P), Shear Wave (S) and Surface Wave (R). These waves induce oscillating ground motion which is experienced as an earthquake and structure standing on ground are subjected to vertical, horizontal and torsional vibrations. These effects are felt the most near the disturbed mass of earth which has ruptured, and decrease with the distance from it. This effect is represented by Equation 6.1, adopted from the work of Gutenberg Richter[53]

$$a = C.\ (E)^{1/2}\ h/(D^2 + h^2) \qquad \text{---} \quad \text{Eqn. (6.1)}$$

where a = ground acceleration expected at D miles from the centre of disturbed mass (distance measured at right angles to the line of rupture in case of epicentre is not a localised one)

'D' is the depth of the centre of disturbed mass below ground surface, 'E' is the total energy released by an earthquake and 'C' is a constant depending upon the nature of soil, geological characteristic of the region etc.

Shri Jai Krishna(83 b) has provided a curve (vide Fig.6.1 attached), correlating magnitude, distance and expected maximum ground acceleration during earthquake to use it as a guide to estimate future ground motion for a shock of certain size. This curve can be used along with another curve of Neumann(84) vide Fig.6.2 (attached) relating to period of vibration and % damping provided by earth dam mass and response of spectrum to decide maximum acceleration for a particular intensity of earthquake shock for an earth dam construction in the region. An illustration as below will reveal its use fully.

Using Fig.6.2 and multiplying factor proposed by Neumann for a structure having a period of 2 seconds vibration and 20 % damping, the response from spectrum curve is 1.8 ft/sec^2 (approx). Supposing structure is located 40 miles from the active fault trace and focus is likely to be 15 miles (an average value for Indian Earthquakes) below ground the maximum ground acceleration from Fig.6.1, for earthquake of magnitude "8" is expected as 55 % g. Comparing this with 33 % standard earthquake shock of El Centro, May 18, 1940 (for which curve in Fig.6.2 holds good viz. for California shocks) we have the multiplying factor as $\frac{55}{33}$ x 2.7 = 4.8

Hence, structure may have to experience a maximum acceleration of 4.8 ft/sec^2 = 16 % g in such a region illustrated above against which additional safety has to be provided to an earth dam structure. In above illustration, response spectrum core

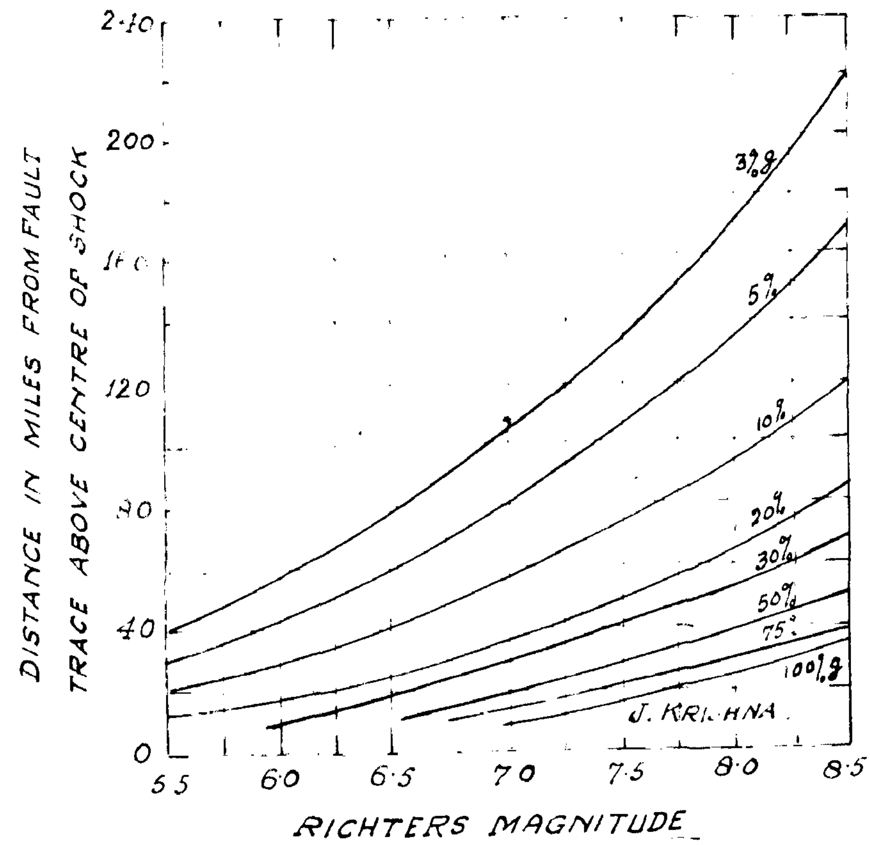


FIG 6·1

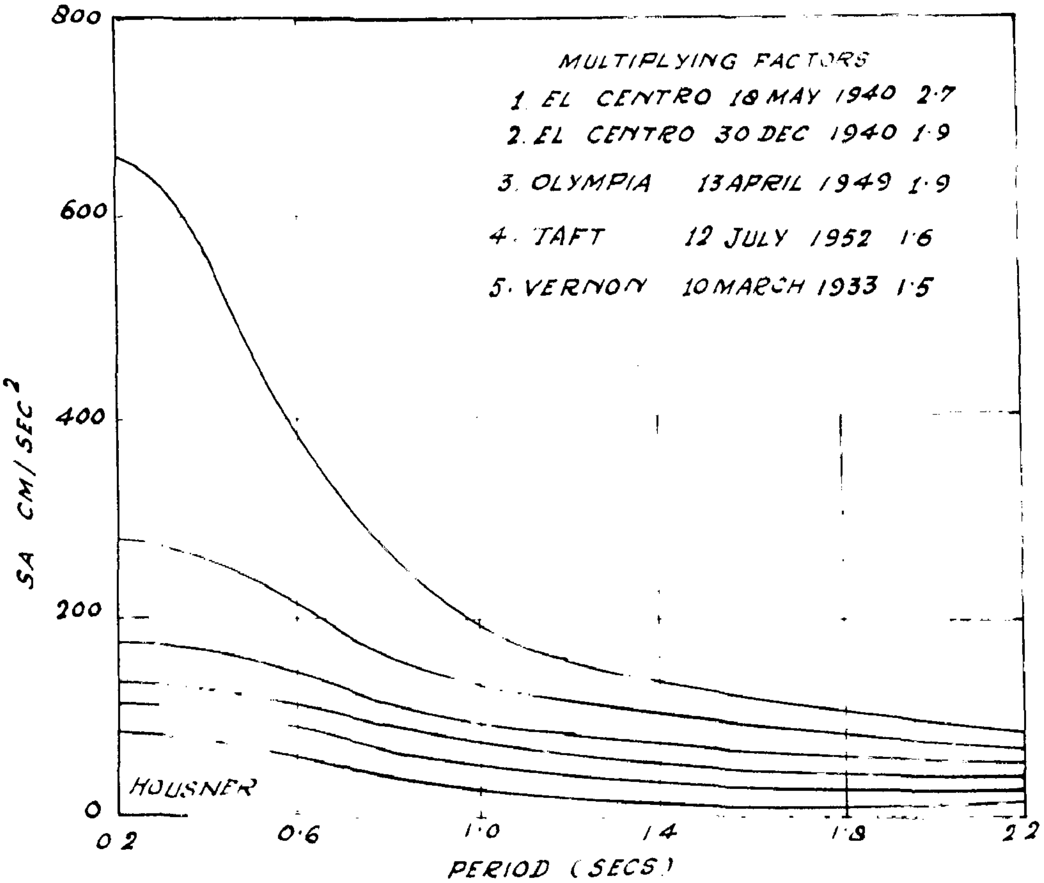


FIG 6·2 AVERAGE ACCELERATION SPECTRUM CURVES gives integrated effect of the ground motion over the period of time, the shock lasts. This 1.2 ft/sec^2 is the integrated effect of acceleration over the whole height of earth dam whereas 4.0 ft/sec^2 is the maximum acceleration to be experienced at top. of Dam

Above analysis requires study of the region, where an earth is to be located and the study should be made in following contents:-

(i) Distance of proposed dam sites from epicentres of live faults in the region (if the site falls in the seismic region)

(ii) Maximum magnitude of earthquakes occuring in the region during known time of history say 100 years, their frequency and disaster and amplitude. The more the informations are gathered, the better.

(iii) Damping characteristics of earth foundation.

THEORY

Supposing waves emanating from the disturbed mass can be represented by Eqn. 6.2 and the intervening strata have elastic properties

$$x = A \sin \frac{2\pi}{T} t \qquad \ldots \text{(Eqn. 6.2)}$$

where x = displacement of soil particle due to wave formation

t = the time interval from initial impulse,

T = the period of wave

A = A constant depending upon size of initial impulse

Then $$\frac{dx}{dt} = A \left(\frac{2\pi}{T} \right) \cos \frac{2\pi}{T} t \qquad \ldots \text{(Eqn. 6.3)}$$

$$\frac{d^2x}{dt^2} = -A \left(\frac{2\pi}{T} \right)^2 \sin \frac{2\pi}{T} t \qquad \ldots \text{(Eqn. 6.4)}$$

Hence maximum velocity = $A \left(\frac{2\pi}{T} \right)^2$ $\ldots$ (Eqn. 6.2a) and maximum acceleration (a) = $A \left(\frac{2\pi}{T}\right)^2$ ... (Eqn.6.4a)

INFERENCES

(1) Eqn. (6.1) indicates that the acceleration caused by an earthquake decreases rapidly with distance.

(2) Eqn. (6.4a) reveals that a short period wave (T) will cause bigger acceleration than a long period wave for the same magnitude of energy transmitted in the earthquake.

(3) Also, Kinetic Energy in above case (Simple Harmonic motion) can be represented by following Eqn.(80)

$$K.E. = \frac{W a^2}{2g(4\pi^2 n^2)} \qquad \text{... (Eqn.7)}$$

Where W = Wt of the body

a = acceleration

n = number of complete vibrations per second

Eqn.7 indicates that K.E. varies directly with the square of the acceleration and inversely with the square of the frequency. This leads to a conclusion that acceleration alone is not a sufficient measure of the destructive power of vibration, and both of acceleration and frequency of acceleration should be studied together in evaluating likely effects of earthquakes. This is discussed below in detail.

A short period wave provides displacement in opposite directions in such a quick succession that the dam and its foundations are likely to get compacted and thereby settle, causing reduction in free board of dam. Hence, to provide safety against failure of earth dams by overtopping enough free board should be provided especially in a region close to epicentre. This does not cause much anxiety to Engineers as they can always do it. On the contrary, long period waves, appearing at further distances present greater cause for anxiety in that the most severe failure could result from chances of resonance. However, since the acceleration caused by an earthquake dies out rapidly from epicentre, Eqn. (6.1), it can be concluded that the long period waves, most likely to affect stability of high earth dams would be confined to a certain zone, at a certain distance from the epicentre of earthquake distance and its width will be dependent upon size of earthquake (magnitude) and characteristics of strata overlying epicentral tract. Other inferences are:-

(4) The softer the soil, quicker will be absorption of the shock wave and hence, area close to epicentre will be most affected by earthquakes, and its width would also be small. Hence, alluvium plains in close proximity of an epicentre can be said to be more vulnerable by earthquakes than hilly tracts in the context of shearing effect of dams. Apart from that, this may cause greater compaction of the foundation, causing greater settlement. Dam itself will be less critical as regards its consolidation.

(5) Earth dams built on semi-rocky foundation 10-30 miles away from the fault will be consolidated more and chances of overtopping will increase. In the next 20 miles or so, the greater danger will be the shearing of the slices from the two slopes. Beyond 40-50 miles away from the epicentre of the earthquake, earthquake force will be considerably smaller, unless the size of earthquake is of the order (of magnitude) of 8 and higher. In region where earthquakes of size 8 or even higher is anticipated the affected region may reach even beyond 60-70 miles in width.

Following counter-measures must therefore be taken into consideration of high earth dam in seismic regions:-

(a) Provide additionally for earthquakes in free board of earth dams to avoid failure by overtopping. Free board may be increased by [illegible] to [illegible] of normal free board.

(b) Earth dams should be thoroughly compacted at the time of construction though there is evidence to show that such dams consolidate still further due to dynamic forces during earthquakes than static forces applied during construction. Even cohesive soils or loess are also found to settle under vibratory condition.

(c) Use construction materials of high shearing strength available near the construction site. This minimises chances of failure, due to slipping of slopes or causing longitudinal cracks.

(d) Flatten the slopes of earth dams. This also minimises chances of slopes slipping or longitudinal cracks.

(e) Avoid constructing high earth dams right across the live faults. This will avoid chances of shearing off of the dam.

(f) Use inclined impermeable core instead of vertical core. During earthquakes, when dams vibrate transversely, the elastic properties of the core and shell materials being different from each other, they vibrate out of phase and there is tendency to separate out at the junction. In this eventuality, the sloping core by virtue of one mass resting over the other, prevents the tendency of separation.

(g) Anchor the dam at abutments properly with hill sides. Still, the junction being a vulnerable point, because earth dam and hill side may vibrate out of phase, owing to these different vibration characteristics, junction should be built with materials having high shearing strength. This provides higher resistance to resonance also.

(h) Sliding of the dam at base is a possibility, during earthquakes of higher magnitude. This should be especially tested during stability analysis of earth dam sections. Unevenness at the junction and the base well anchored to foundation may provide extra safety against such sliding.

(i) Consolidate pervious foundation, before constructing earth dams if there are chances of liquefaction of fine sand in the foundation during earthquakes.

(j) Donot embed outlet structures with the body of dam. They should be located preferably separately in on flanks or saddles.

6.1.9. I - <u>LIQUEFACTION OF FINE SAND</u>

All except 9th counter-measure as suggested in para 6.1.8 can be adequately met with by construction engineers at site, while counter-measure of consolidating pervious foundation forming fine sand is not very common and hence requires collaboration of Geologist, Seismologist and Soil MechanicsExperts to decide whether the in-place sand is likely to liquefy under the existing set up of condition and expected intensity of earthquake during life time of the structure (say 100 years). Details of two such liquefaction tests in India are available, viz. of Gora and Tenughat sand - and they are summarily presented below:-

IX - THEORY OF LIQUEFACTION[56]

Phenomenon of Liquefaction of soil is a consequence of increase in pore pressure, due to upward hydraulic gradient which may be caused by dynamic loading of earthquake. The reduction in shear strength in this case depends on among other factors, the grain size of sand and the density of deposit. In fine sand the reduction in shear strength may be complete whereas in coarse sand it is mechanical properties of sand like grain size, uniformity coefficient, permeability, porosity etc. only which would make them susceptible to measurement[60]. However, later investigations have shown that these are not the only factors but the characteristics of dynamic load such as amplitude, frequency and direction of motion are also important. It is now felt that the phenomena is a result of the collapse of the structure of sand which causes a sudden rise in pore pressure that cannot be dissipated immediately and lead to liquefaction.

We know that for sands

$$s = (\sigma - u) \tan \phi \qquad \text{(Eqn. 6.5)}$$

where s = Shear strength

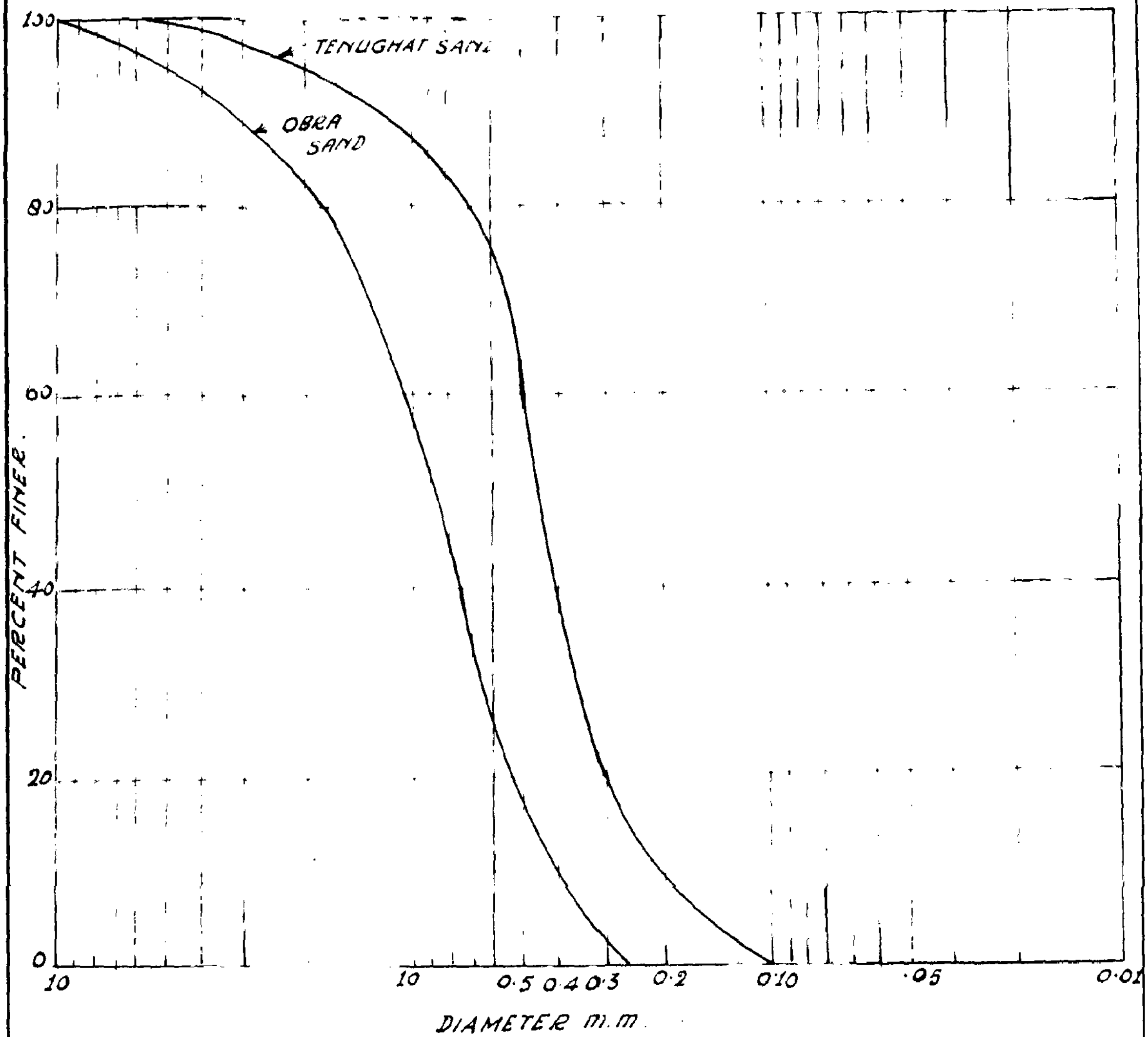
σ = Total normal stress = $(\gamma \cdot z + w\,h)$ (Vide Fig.6.3)

u = Pore pressure

ϕ = Angle of internal friction

$\therefore$ $s = \bar{\sigma} \tan \phi$ where $\bar{\sigma}$ is effective stress $(\sigma - u)$ Eqn. 6.6

If additional pore pressures are developed because of dynamic load, strength becomes



GRAIN SIZE DISTRIBUTION

FIG. 6·4

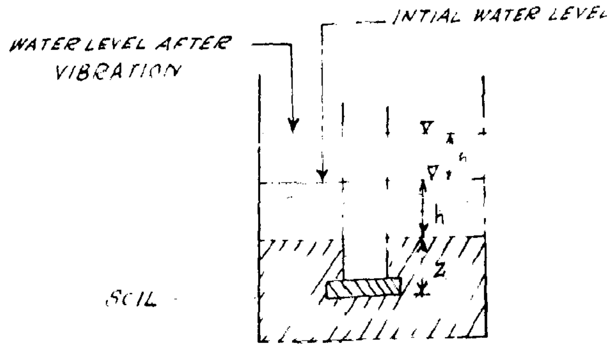


FIG. 6·3

(8) $\tau_n = (\bar{\sigma} - \Delta u) \tan \phi$ (Eqn.6.7)

where u = Additional pore pressure = $\gamma_w . h'$, h' being the additional head of water due to dynamic loads.

When $\bar{\sigma} = \gamma_w \cdot h'$ shear strength is zero and the soil will behave like fluid. This is defined as liquefaction. The upward hydraulic gradient for this stage is given by

$$\frac{h'}{h} = \frac{\gamma_b}{\gamma_w} = \frac{G_s - 1}{1 + e} = i_{cr} \qquad \text{(Eqn.6.8)}$$

where γ_b = Submerged unit weight of sand

G_s = Sp. Gr. of soil solids

e = void ratio

i_{cr} = Critical hydraulic gradient

If the void ratio of soil is close to 0.6 and G_s close to 2.6 $i_{cr} = 1$ and $h' = h$ i.e. for liquefaction to take place the additional pore water head equals the depth at which liquefaction occurs.

XII – BLASTING TESTS AT OBRA AND TENUGHAT SITES

Blasting Tests were carried out at Obra site in the year 1966 and at Tenughat in the year 1967, for following purposes-

(1) To determine if liquefaction of Obra sand & that at Tenughat occurs under simulated earthquake loading, and

(2) To compare the field results with those obtained in the laboratory earlier[80] and

(3) To ascertain if compaction of the Obra and Tenughat sand can be conveniently completed by blasting

IV - <u>CHARACTERISTICS OF OBRA AND TENUGHAT DAM SITE</u>

Followings are characteristics of Obra & Tenughat sands

Characteristics	Obra Sand	Tenughat Sand Vide Fig.6.4 for granding cover)
1. Sp. Gr. of grains	2.61	2.62
2. Maximum void ratio e_{max}	67.8 %	79 %
3. Minimum void ratio e_{min}	40 %	49 %
4. Dry Density d_{max}	-	1.76 gm/c.c
5. Dry density d_{min}	-	1.46 "
6. Uniformity coefficient Cu ($= \frac{D_{60}}{D_{10}}$)	5.45	2.37
7. Coefficient of curvature $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}}$	3.45	-
8. Soil according to IS 1498 - 1959 (Requirement for (SP) C_u 4, C_c = 1-3	(SP)	(SP)
9. Relative density	12' depth - 30 % 40' depth - 50 % 28' depth - 70 %	Not available

Inference:- Tenughat sand is approximately as coarse as Obra sand but e_{max} and e_{min} are both quite higher than that of Obra.

V - [illegible] BLASTING

Scheme of blasting along with a site plan are shown in Fig.6.8 (a) & (b). Charge, used for blasting was 60 % of gelatine in all cases

Quantity of charge placed at 6.0 m depth	Distance from Observation post		Acceleration - Observation
1 Kg Charge	(a)	10 m	Two acceleration pickups were placed about 20 cm below the ground in such a manner that one of these recorded horizontal acceleration and the other vertical acceleration
	(b)	20 m	
	(c)	30 m	
	(d)	50 m	Result : Fig.6.9 (a) $\alpha_V \doteq 2\,\alpha_H$
2 Kg charge	(a)	10 m	-do-
	(b)	20 m	
	(c)	30 m	
	(d)	10 m (Charge fired twice)	(Result : $\alpha_V = \alpha_H$ Fig.6.9 (b)
3 Kg charge	(a)	30 m	Result Fig.6.9 (c)
	(b)	50 m	Max. V = 0.90 (interpolated) H = 1.06 -do-

N.B. One 3 Kg charge fired from a depth of 4 m. caused the ground to heave, an apparent case of liquefaction.

VI - [illegible] OF ACCELERATION

From the observed values, curves of the form

$$a = C \frac{Q^{\alpha}}{d^{\beta}} \quad \text{were fitted} \qquad \text{(Eqn.6.9)}$$

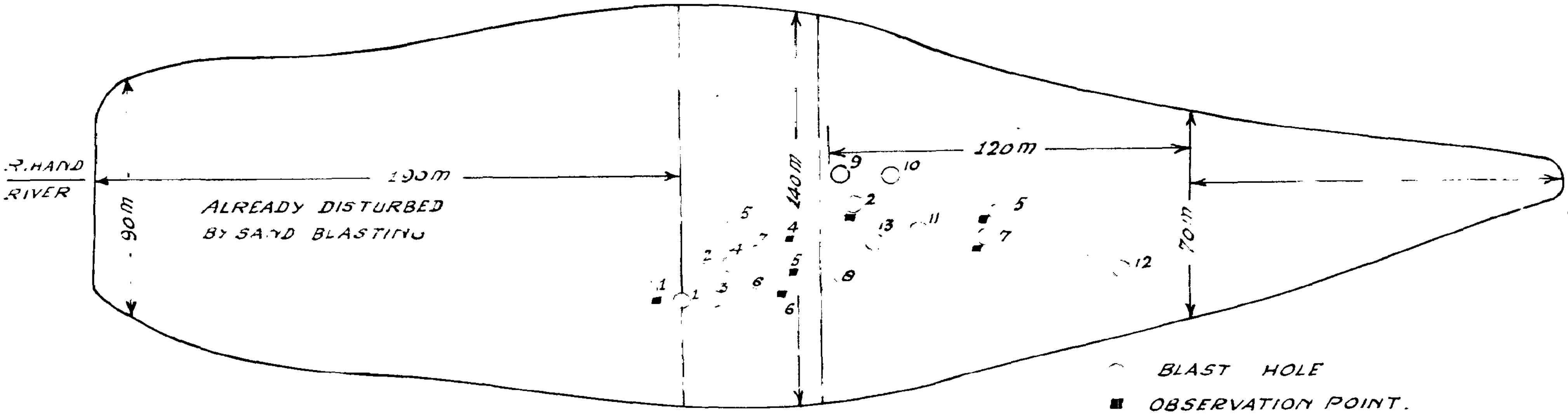


FIG. 6.5(a) SITE PLAN OF BLASTING AREA

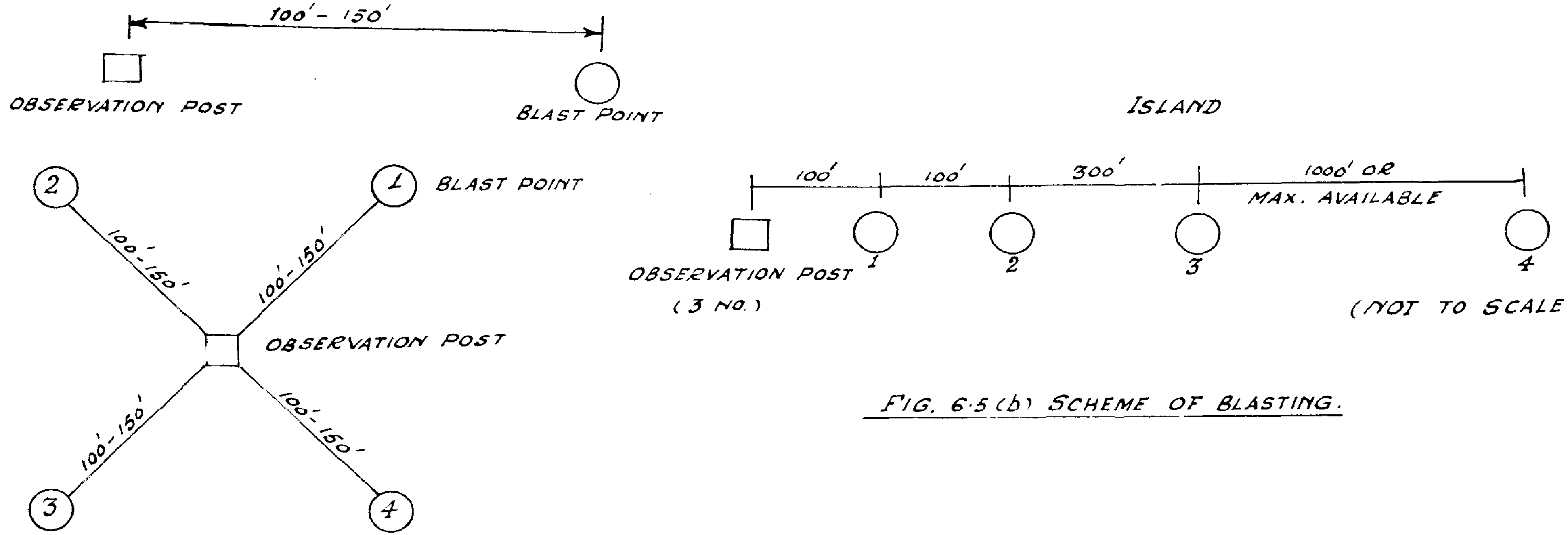


FIG. 6.5(b) SCHEME OF BLASTING.

where a = Acceleration in 'g'

Q = Amount of charge in Kg

d = Distance of charge point from observation point in metres

C, α, β = Constants

For charges fired at 6 m depth, it was found that

	C	α	β
For Horizontal Acceleration	27	0.07	1.20
For Vertical Acceleration	75	0.07	1.20

The values of acceleration was computed with the help of Eqn.(6.9) established for vertical and horizontal acceleration and the curves shown in Fig.6.6 are based on this equation. Maximum values of α_V & α_H for 2 Kg blasts were 6.00 g and 1.04 g respectively.

[illegible]

Values of acceleration are close to the computed ones except in a couple of cases (see Fig.6.6)

LIQUEFACTION OF OBRA SAND

Pore pressures and settlement observations at the time of blasting are given in Table No.6.3 with 2 Kg blast. Change in pressure in the Table is given as %age of initial intergranular stress[30]. From this table maximum observed pore pressure is 63 % of the intergranular stress at a depth of 3.5 m and at a distance of 2.8 m from blast point. By interpolation from the Curve Fig.6.7 at a point of blasting this maximum change in pore pressure is found to be about 80 % intergranular pressure. For the Obra sand to liquefy this pore pressure should equal the initial intergranular stress by 100 % which it did not even at α_V = 6.00 g & α_H = 1.06 g respectively against anticipated α_V & α_H = 0.1 g for the region. Hence it was concluded that Obra sand will not liquefy. Only knock

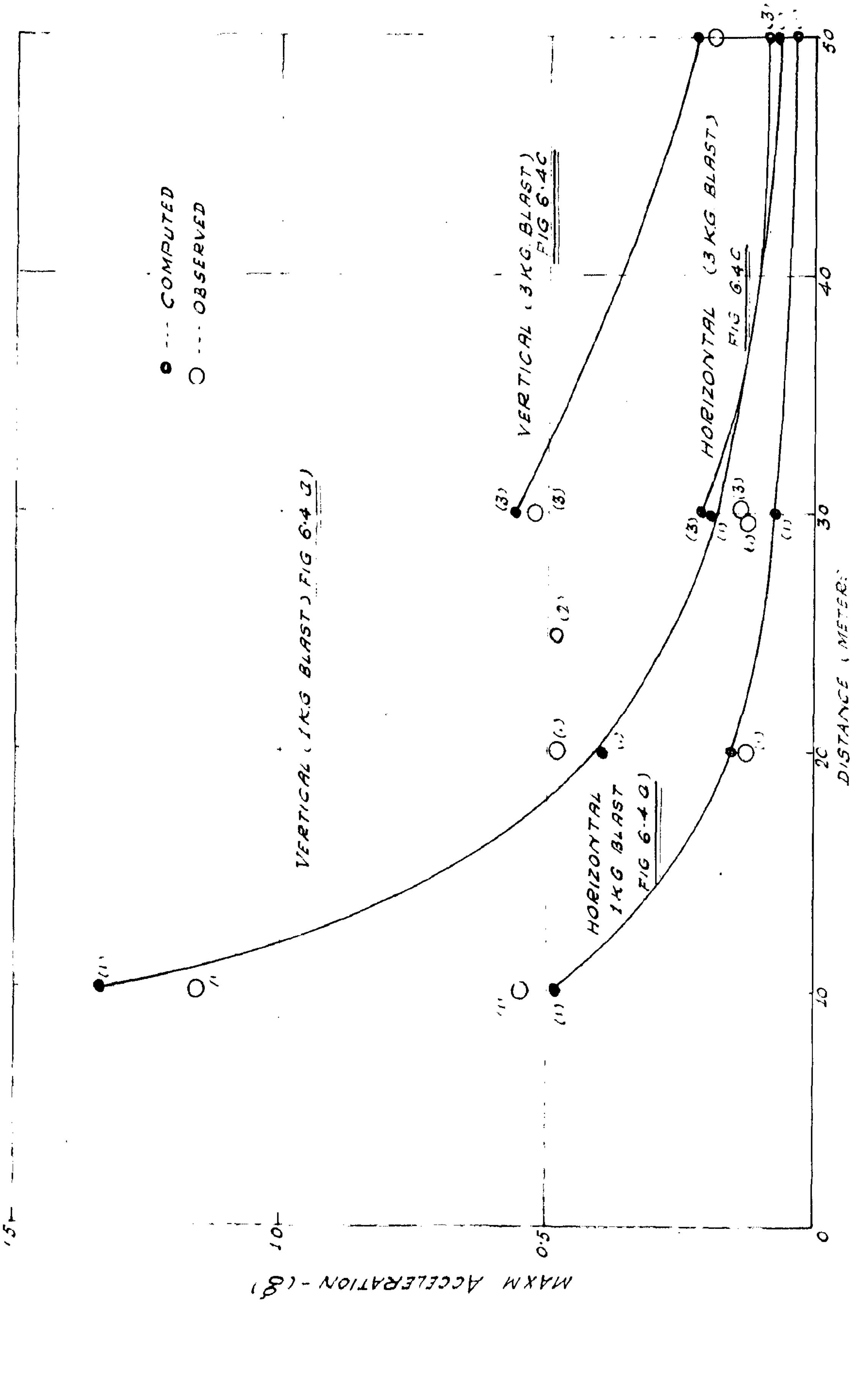


FIG. 6.6.1 ACCELERATION VS. DISTANCE (1 KG. AND 3 KG. BLAST)

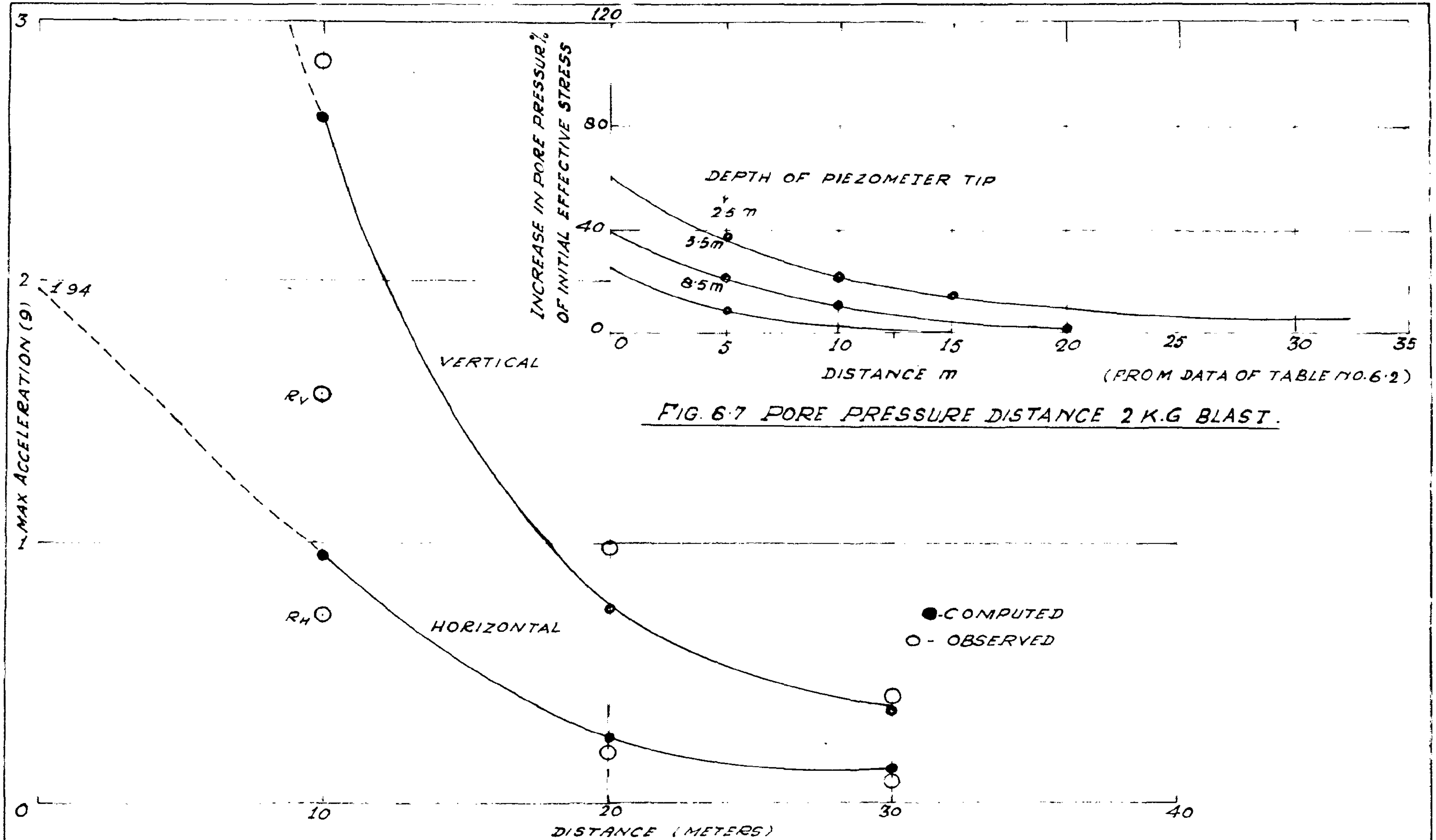


FIG. 6.7 PORE PRESSURE DISTANCE 2 K.G BLAST.

FIG. 6.6 (b) ACCELERATION VS DISTANCE 2 K.G BLAST in the conclusion was that the test was done with a single blast at a time and hence it did not simulate the condition of actual earthquake tests. Jai Krishna and Prakash(67) therefore supplemented these tests results by laboratory, simulating earthquake as number of cycles and finally concluded as below:-

(1) Obra Dam sand is not likely to liquefy under anticipated ground motion.

(2) The field data are in conformity with the laboratory data already obtained and vice-versa

(3) Compaction of the Obra sand may not be necessary

(4) A flexible apron 3 m or so thick graded as inverted filter material may be provided at upstream and downstream toes for a length of about 30 m.

TABLE NO. 6.2

MAXIMUM INCREASE IN PORE PRESSURE AFTER THE BLASTING OF [illegible] (OBRA)

Distance from centre of blast → / Depth of piezometer ↓	2.5 m		5 m		10 m		15 m		30 m	
	In cm	In % of effective mean	In cm	In % of effective mean	In cm	In % of effective mean	In cm	In % of effective mean	In cm	In % of effective mean
2.0 m	170	68	132	37	60	10	40	14	31	6.0
3.0 m	181	30	130	20.8	53	6.7	18	2.0	0	0
6.8 m	140	15.2	60	8.8	10	1.1	0	0	0	0

VI - TENUGHAT SAND BLASTING

Proportion of Tenughat sand and seismicity of Tenughat region being more or less the same as that of the Obra, it was thought that Tenughat sand also might not liquefy under 10 % of (Zone 2, Vide Fig.6.10) acceleration caused by earthquakes of intensity 6 < as decided upon by Meteorological Department of Government of India. This was confirmed by actual blast tests at Tenughat site in February 1969. Results of blasting tests for acceleration in vertical and horizontal directions, maximum increase in pore pressure at observation points, settlement and penetration resistance observed at Tenughat site are tabulated here in the Table No.6.3 to 6.6.

Maximum acceleration observed in different cases are noted in Table No.6.3. In case of three successive blasts of ½ kg each it was noted that the vertical acceleration (a_v) went to 1.0 g while horizontal acceleration (a_H) reached 0.30 g only. Against this maximum observed % effective stress was noted as 70.8 % at a depth of 6 m and at a distance of 3 m from blast point. Corresponding settlement came as -0.01 to -1.33 at 6 m depth (Table No.6.5). Static penetration test result can also be seen at (Table No.6.6). It can be further seen from the appended tables that in case of four successive blasts, the % effective stress reached as high as 100 % and that the sand was apparently forced to liquefy. Tenughat Dam site being under seismic intensity of 6 on the Modified Mercalli Scale - 1931, (which tallies with Cora sand also of magnitude being (6 - 6.5). 10 % g was considered reasonable for design or test purposes. At this acceleration, Tenughat Sand was not found to liquefy, from blast tests and confirmed also by laboratory tests at School of Earthquake Research and Training Roorkee, later.

TABLE NO.6.3 (TENUGHAT)

HORIZONTAL VERSUS VERTICAL ACCLERATION

Blast No (1)	Charge Quantity (kg) (2)	Depth (m) (3)	Distance from blasting point (m) (4)	Accelertion Depth of Pickup (5)	Horizontal Acceleration (g) (6)	Vertical Acceleration (7)
			REPEAT BLAST			
$B_1(P_1)$	1	6	30	Surface	0.2 g	0.4 g
$B_2(P_1)$	1	6	30	Surface	0.2 g	0.25 g
$B_9(P_1)$	1	6	30	Surface	0.16 g	0.4 g
			DEPTH VS ACCELERATION			
$B_3(P_1)$	1	4	20	Surface	0.32 g	1.3 g
$B_4(P_1)$	1	6	20	Surface	0.32 g	0.88 g
$B_6(P_1)$	1	9	20	Surface	0.70 g	1.7 g
			CHARGES VS ACCELERATION			
$B_8(P_1)$	½	6	20	Surface	0.17 g	1.1 g
$B_4(P_1)$	1	6	20	Surface	0.32 g	0.86 g
$B_7(P_1)$	2	6	20	Surface	0.5 g	1.4 g
			DISTANCE VS ACCELERATION (CHARGE ½ KG)			
$B_{12}(P_1)$	½	6	10	Surface	0.5 g	1.3 g
$B_9(P_1)$	½	6	20	Surface	0.12 g	1.1 g
			DISTANCE VS ACCELERATION (DEPTH OF CHARGE 4 m)			
$B_{10}(P_1)$	1	4	10	Surface	0.6 g	2.0 g
$B_3(P_1)$	1	4	20	Surface	0.32 g	1.3 g
			DISTANCE VS ACCELERATION (DEPTH OF CHARGE 6 m)			
$B_9(P_1)$	1	6	10	Surface	0.8 g	2.0 g
$B_4(P_1)$	1	6	20	Surface	0.32 g	0.86 g
$B_1(P_1)$	1	6	30	Surface	0.2 g	0.4 g
$B_{13}(P_1)$	1	6	50	Surface	0.06 g	0.1 g

TABLE NO. 4.3 -(Contd)-

(1)	(2)	(3)	(4)	(5)	(6)	(7)
			DISTANCE IN ACCELERATION (DEPTH OF CHARGE 2 m)			
$D_{11}(P_1)$	1	0	10	Surface	10.5 g	3.0
$D_6(P_1)$	1	0	20	Surface	0.7 g	1.7
			DEPTH OF PICKUP IN ACCELERATION			
$D_0(P_1)$	1	0	10	Surface	-	2
$D_4(P_2)$	1	0	10	3 m	-	4.4
			DEPTH OF PICKUP IN ACCELERATION (DISTANCE FROM THE BLASTING POINT) 20 m			
$D_1(P_1)$	1	0	20	Surface	-	0.6
$D_1(P_2)$	1	0	20	3 m	-	2.0
			NUMBER OF CYCLES OF ACCELERATION			
$D_0(P_1)$	4	0	20	Surface	0.17 g	1.1
$D_0(P_2)$	4	0	20	-do-	0.5 g	1.0
3 successive blasts	4 kg each	0	20	-do-	(i) 0.06	0.33
					(ii) 0.20	1.0
					(iii) 0.20	1.0

TABLE NO.6.4

TENUGHAT BLASTING TEST

MAXIMUM INCREASE IN PORE PRESSURE AFTER BLASTING OF DIFFERENT CHARGES

Plot No	Blast No	Depth of Charge (m)	Size of Charge (Kg)	Distance of piezo meter from blast point in m	RISE IN PORE PRESSURE: At 3 M depth (2.54): 0 Minutes	Eff. Pr.	% PP	At 6 M depth (5.54): 0 Minutes	Eff. Pr.	% PP	At 9 M depth (8.54): 0 Minutes	Eff. Pr.	% PP
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
	B_1 (Repeat)			3	1.74 m	3.11	56.0	3.88 m	6.02	64.4	3.18 m	8.92	35.6
	17.3.67 9.55 A.M.	6	1	6	1.16 m	3.15	36.7	2.41 m	6.06	39.7	2.66 m	8.95	29.7
				15	0.305 m	2.94	10.4	1.22 m	5.83	20.9	0.546 m	8.75	6.25
				30	0.076 m	2.78	2.7	0.292 m	5.68	5.1	0.343 m	8.58	4.0
	B_2			3	2.09 m	3.06	68.00	4.70 m	5.99	78.5	4.32 m	8.88	49.8
	17.3.67 11.05	6	1	6	1.45 m	3.13	46.3	1.96 m	6.03	32.5	3.04 m	8.93	34.0
				15	0.432 m	2.92	14.8	1.35 m	5.81	23.2	0.585 m	8.73	6.7
				30	0.051 m	2.76	1.8	0.279 m	5.66	26.2 4.9	0.216 m	8.58	2.52
	B_3			3	2.10 m	3.03	69.0	4.51 m	5.95	75.8	4.03 m	8.85	45.5
	17.3.67 12.40	6	1	6	1.475 m	3.13	47.0	1.755 m	6.01	29.2	3.34 m	8.91	37.4
				15	0.483 m	2.92	16.5	1.245 m	5.81	21.4	0.649 m	8.73	7.4
				30	0.019 m	2.76	0.7	0.203 m	5.66	3.6	0.222 m	8.58	2.6
I	B_4	6	1	20	0.267 m	2.85	9.4	0.615 m	5.86	10.5	0.022 m	8.64	0.25
	B_5	4	1	20	0.216 m	2.85	7.6	0.133 m	5.86	2.26	0 m		0
	B_6	9	1	20	0.137 m	2.85	4.8	0.306 m	5.84	5.2	0 m		0
	B_7	6	2	20	0.610 m	2.85	21.4	1.626 m	5.86	27.7	0.33 m	8.64	3.8
	B_8	6	½	20	0.0762 m	2.85	2.7	0.0732 m	5.86	1.25	0.0127 m	8.64	0.15
	B_9	6	1	10	1.230 m	2.85	43.0	2.48 m	5.86	42.2	2.20 m	8.64	25.5
	B_{10}	4	1	10	1.335 m	2.85	46.8	2.62 m	5.86	44.6	1.99 m	8.64	23.0
	B_{11}	9	1	10	0.813 m	2.85	28.5	1.88 m	5.86	32	1.805 m	8.64	20.9

TABLE NO.6.4 -(Contd)-

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
II	1	6	1	20									
	2	6	1	10									
	3	6	1	20									
	4	6	1	10									
III	3 Blasts in quick succession with a time interval of 0.5 to 1 sec	6	½ Kg each	3	3.22 m	2.58	12.5 %	3.87 m	5.50	70.5	2.20 m	8.40	26.2
				6	1.435 m	2.58	58.5 %	2.44 m	5.50	44.5	2.52 m	8.40	30.0
				15	0.033 m	2.60	1.2	-0.05 m	5.50	0	0.983 m	8.40	11.3
				20									
	4 Blasts	6	1 Kg each	3									
				6	3.0 m	2.60	120	5.75 m	5.40	106	4.96 m	8.30	6.0
				15	0.94 m	2.50	-	8.20 m	5.40	59	3.20 m	8.30	38.6

Eff. Pr. = Effective Pressure in water height $\bar{\sigma} = \sigma - p$

% PP = Pore Pressure in % of effective stress

$p_1 - p_2 = \bar{\sigma}_2 - \bar{\sigma}_1$

$\bar{\sigma}_1 = \sigma_1 - p_1$

$\bar{\sigma}_2 = \sigma_2 - p_2$

TABLE NO. [illegible]

SUMMARY OF SETTLEMENT DATA (CONTINUED)

Plot No	Blast No	Charge (kg)	Depth (m)	Depth of settlement Plate	Settlement at 3 m distance in cm	Settlement at 6 m distance in cm	Settlement at 10 m distance in cm
I	B_1	1	6	Surface 6 m depth	6.63 19.3	3.0 -3.39	Nil
	B_2	1	6	Surface 6 m depth	5.6 6.25	2.78 2.44	0.92
	B_3	1	6	Surface 6 m depth	3.03 6.10	2.44 2.44	1.63
	B_4	1	6	Surface	5.20	4.27	0.3
	B_5	1	4	Surface	6.10	2.74	1.03
	B_6	1	6	Surface	-0.01	-2.74	-3.30
	B_7	2	6	Surface	7.33	3.66	-0.01
	B_8	4	6	Surface	3.35	0.92	0.01
	B_9	1	6	Surface			
	B_{10}	1	6	Surface	6.25	3.60	1.28
	B_{11}	1	6	Surface	7.33	1.63	0.01
II	B_1	1	6	Surface			
III	4 cycles	1	6	Surface	9.43 6.05	4.88 7.02	-1.03
	3 cycles	4	6	Surface 6 m	-0.01 -0.01	-1.22 -2.13	-1.22

-ve sign shows heaving

TABLE NO. 6.8

SUMMARY OF THE STATIC PENETRATION DATA

Plot No	Blast No	Charge (Kg)	Depth (m)	3 m distance	6 m distance	15 m distance
I	B_1 B_2 B_3	1 Kg each	6	0-3.25 M 22-31 kg/cm^2 Increase 41 % 3.5 - 4.5 M 53 - 38 kg/cm^2 Decrease 28 %	4 - 7.0 M 48-37 kg/cm^2 Decrease 23%	3.8 - 7 M 57 - 49 kg/cm^2 Decrease 14 %
	B_4	1	6	3.5 - 8 M 42-26 kg/cm^2 Decrease 38 %	2 - 8.0 M 46-32 kg/cm^2 Decrease 30 %	1.8 - 7.0 M 21-45 kg/cm^2 Increase 115 %
	B_5	1	6	2-6 M 50-35 kg/cm^2 Decrease 30 %	3 - 7.8 M 20-40 kg/cm^2 Increase 50 %	0-2.5 M 18-29 kg/cm^2 Increase 60 % 2.8 - 5 M Increase 25% 4-9 M Decrease 25 %
	B_6	1	9	0-6 M 15-33 kg/cm^2 Increase 120%	0-6 M 18-30 kg/cm^2 Increase 66 %	4-6 M 65-43 kg/cm^2 Decrease 33%
				7-9.8 M 70-50 kg/cm^2 Decrease 37 %		
	B_7	2	6	0-3.25 M 21-37 kg/cm^2 Increase 76 % 6-9.5 M 61-43 kg/cm^2 Decrease 30 %	0-6 M Increase about 30 %	0-6 M 25-36 kg/cm^2 Increase 44 %
II	3 successive blasts	4 Kg	6	Practically no change	2-5.8 M 39-23 kg/cm^2 Decrease 41 % 5.8 - 8.8 M 16-23 kg/cm^2 Increase 44 %	3.8 - 7.8 M 81-51 kg/cm^2 Decrease 37 %

6.2. RECORDING CONSTRUCTION PORE PRESSURE

6.2.1. INTRODUCTION

During construction of an earth dam in layers, water content of soil fill (either coming in natural quantity from borrow area or mixed at site) gets trapped inside the pores of the soil fill and exert pressure on the body of the soil grains, known as pore pressure or construction pore pressure. If construction is too rapid, such pressure is not able to dissipate away during construction period itself and continues to dissipate much after construction is over. This pore pressure being neutral stress reduces the normal effective stress but increases the tangential pressure endangering the stability of earth dam. Hence, designers have to account for it and check the stability of dam for construction pore pressures also, based on theoretical analysis or laboratory tests. Hilf[63,7] has given an equation for pore pressure combining Boyle's Law for compression of gases and Henry's Law for solubility of gas and assuming the soil to be non-draining. Bishop[64,65] and Li[66] have given an approximate method for estimating pore pressure, with a dissipation factor. This factor accounts for a certain %age of pore pressure which is actually dissipated during construction period. But apart from all these theoretical approach to the problem, actual observation of such pore pressure during and after construction of the earth dam and efforts to control them would alone provide a practical safe-guard to the structures. India has also taken a lead in this direction by collecting a large number of informations on construction pore pressures of important earth dams. But this task has to be done in future with more determination and planning.

6.2.2. ACTUAL OBSERVATIONS

Nanak Sagar Dam (Nainital) and Moosakhand Dam (Varanasi) completed in Uttar Pradesh in the year 1962 and 196[illegible] respectively, have been maintaining records of pore pressure and settlement since beginning. At Nanak Sagar Dam placement moisture was 18 % (1 % wet of O.M.C.) while at Moosakhand Dam it was 11 % (3 % dry of O.M.C.). Actual observations have shown that maximum increase in pore pressure in the former case was quite considerable (6 kg/cm^2) than that in other case (0.7 kg/cm^2), the dissipation factor in the two cases being 1/7 and 1/6 respectively. It is, therefore, inferred that construction of earth dams with moisture content on drier side of O.M.C. develops less construction pore pressure and has a lesser dissipation factor than that in case of construction with moisture content on wet side of O.M.C.

Above inference can be confirmed by a good many other examples of Table No.6.7 (attached). In the context of rapid construction of high earth dams, whereas it is desirable to work with a reasonable margin on wet of O.M.C., it is as well necessary to control the excessive pore pressure likely to develop due to wet construction. This control of pore pressure with wet construction has caught the eyes of many, but fact remains that further research on this has got to be done, before perfecting one method or the other.

6.3.3. CONTROL OF PORE PRESSURE

At Usk Dam (Britain) (Fig.6.8) where construction pore pressure was observed to have reached a maximum of 100 % of superimposed load, they provided a drainage blanket over the top of the finished layer of previous season's work and took up construction in the subsequent season. This worked satisfactorily

S.No.	Name of the Dam (outside India)	Pressures Laboratory Value	REMARKS
(1)	(2)	(6)	(7)
1	Rechaupten Dam (Germany)	Maximum Pore Pressure = 80 %	Lab. O.M.C. and density not available to compare with placement moisture content and compaction achieved at site.
2	Sasumua Dam ([illegible] (1949-50)	Vary for different borrow pits	Detailed analysis and graphs available in the Paper (20)
3	Daer Dam (Brit	Laboratory confirmed that no pore pressure will build up since permeability was quite high	Pore pressures usually do not develop on the dry side of O.M.C. to endanger stability of earth dams
4	Usk Dam (Brito		(a) To reduce construction pore pressure a horizontal blanket was inserted within dam at the height of each season's filling (b) Simple stand pipe did not work properly
5	[illegible] Dam (9)		This confirms the remarks in S.No.3 above.

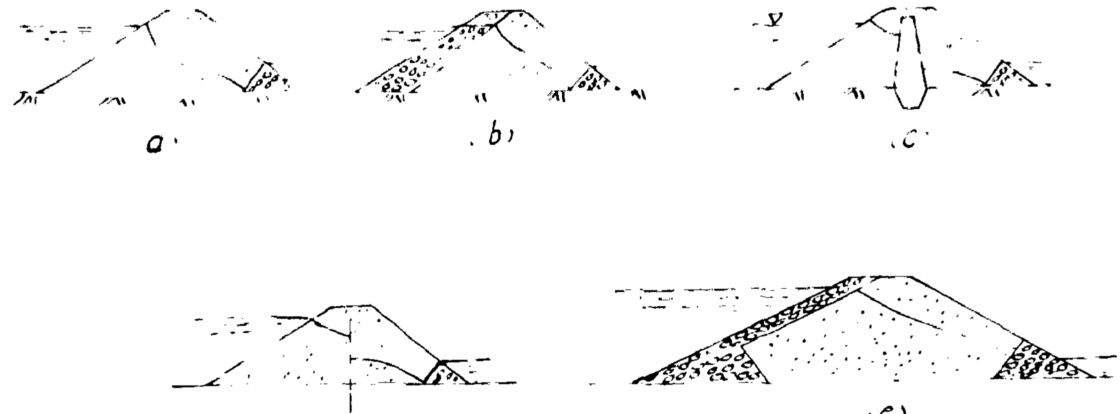
PORE PRESSURE INSTRUMENTATION IN FOREIGN DAMS

TABLE NO. 6.3

S.No. (1)	Name of the Earth Dam (outside India) (2)	Short Description about Dam (3)	Instrumentation (4)	Variation of Pore Pressure: Actual (5)	Variation of Pore Pressure: Laboratory Value (6)	REMARKS (7)
1	Rosshaupten Dam (66) (Germany)	130' (41 m) high dam with a central core of boulder clay and supporting fills of gravelly moraine	Electrical pore water pressure transmitters (DG 70 of firm MAIHAK Hamburg) - 20 Nos installed in 3 tiers in core and upstream impervious blanket. Such pore water pressure meters are also called Electro Acoustic manometers	In the beginning 100 % superimposed load as pore pressure was observed while with construction works going ahead this decreased to (65 % - 66 %) along the height of dam from bottom to top (Observation for 5 years)	Maximum Pore Pressure = 80 %	Lab. O.M.C. and density not available to compare with placement moisture content ~~and construction pore pressure~~ and compaction achieved at site.
2	Sasumua Dam (Kenya) (69) (1953-56)	115' high dam built of residual clay (approx. (LL = 78 %) (PI 38 %) - Compacted at O.M.C.	Piezometer type (30 Nos) Building Research Station Type (An improvement of U.S.B.R. Type) (67)	Maximum initial pore pressure during construction 31 % of superimposed load reduced to zero by end of construction	Vary for different borrow pits	Detailed analysis and graphs available in the Paper (69)
3	Daer Dam (Britain) (70)	130' high - boulder clay compacted at 1.1 % dry of O.M.C.	Piezometer (Building Research Station Type) installed in May 1951 for the first time in Britain	No pore pressure developed	Laboratory confirmed that no pore pressure will build up since permeability was quite high	Pore pressures usually do not develop on the dry side of O.M.C. to endanger stability of earth dam
4	Usk Dam (Britain) (72)	110' high - boulder clay compacted at 2.5 % wet of O.M.C. (9.8 % for 130.6 lb/cft density) - in 12" layers - 95 % compaction achieved (Proctor density)	(a) Piezometer (B.R.S.Type) - 12 Nos. (b) Simple stand pipe - 10 Nos (1952)	Maximum construction pore pressure - 100 % of superimposed load at the end of first construction season and dissipated very slowly		(a) To reduce construction pore pressure a horizontal blanket was inserted within dam at the height of each season's filling (b) Simple stand pipe did not work properly
5	Hongkong Dam (70)	95' high - consisting of lateritic red and white clay compacted at 0.5 % wet of O.M.C. (16.9 % for 100.8 lb/cft density) - 6" thick layers sloping outwards from the core at 1 in 20 - 99 % compaction achieved (Proctor density 100.8 lb/cft	Stand pipes installed and soundings taken at regular intervals	Only small pore pressures were recorded		This confirms the remarks in S.No.3 above.

TABLE NO.6.7 -(Contd)-

(1)	(2)	(3)	(4)	(5)	(6)	(7)
6	Hanning Field Dam[70] No.2 (Britain)	About 30' high - constructed with fat clay 3-7 % wet of O.M.C. (25.21 %) for density 99 - 102 lb/cft. Compaction achieved 95 % of Proctor)	Piezometer (B.R.S.Type) -24 Nos	(-)ve pore pressure recorded		(-)ve pressure due to swelling of fat clay (L.L = 45 % - 68 %) P.I = 17 % - 29 %) may be due to being slightly over-consolidate
7	Foxcote Dam(Britain)[70]	About 30' high - constructed of fat clay and compacted at 1 % wet of O.M.C	Piezometer (B.R.S.Type) -24 Nos	Results similar to that in S.No.6 above		Swelling of clay might have caused (-)ve pressure
8	Serre Poncon Dam[71] (France)	430' high - consisting of slity sand and gravel fill compacted in layers at about 2 % wet of O.M.C.		No details interms of %age of superimposed load available, but it developed high pore pressure during construction		Wet fill construction involves high pore pressure



(a) PERVIOUS FILL WITH A TOE FILTER.

(b) PERVIOUS FILL WITH A TOE FILTER AND U/S PROTECTION BLOCK.

(c) PERVIOUS FILL WITH A THIN IMPERIOUS CORE.

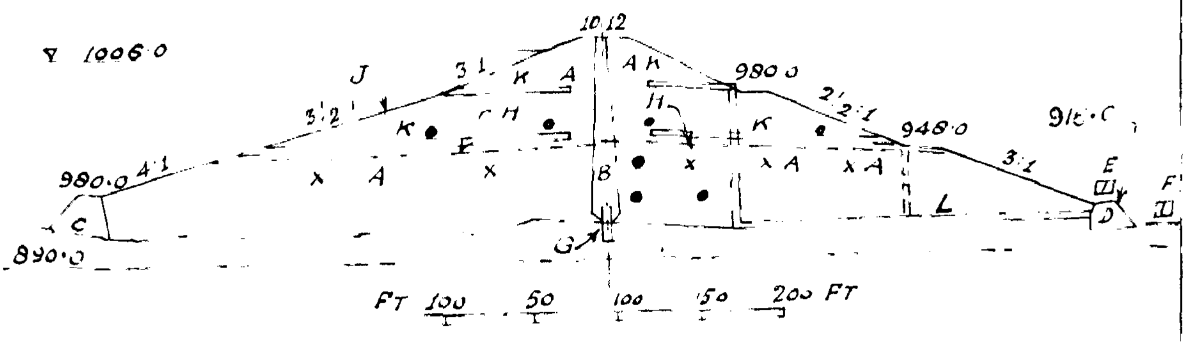
(d) PERVIOUS FILL WITH A DIAPHRAGM AND ROCK TOE.

(e) PERVIOUS FILL WITH U/S AND D/S ROCK TOES AND U/S PROTECTION BLOCK.

FIG. 6·9 DIFFERENT SECTIONS WITH SAND AS CHIEF CONSTRUCTION MATERIAL

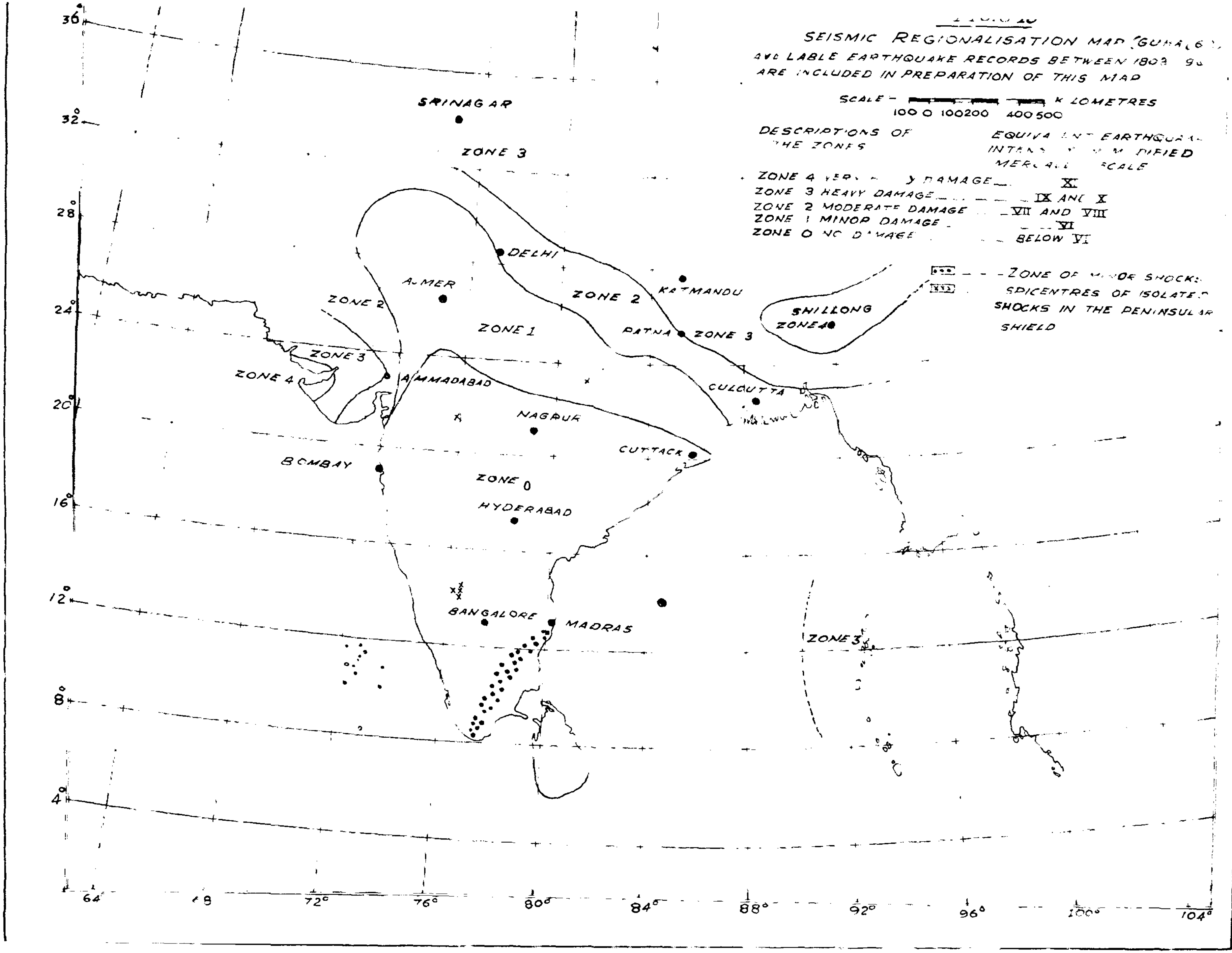
USK DAM

● PIEZOMETER POINTS.
x = STAND PIPE POINTS
A = ROLLED CLAY CORE
B = PUDDLE CLAY CORE
C = U/S ROCK TOE
D = D/S ROCK TOE
E & F = GAUGE HOUSE FOR PORE PRESSURE INSTALLATION
G = CORCETE CUT OFF
H = LEVEL OF CONSTRUCTION AT TIME OF INSTALLATION OF STAND PIPES.
J = U/S CONCRETE SLABBING
K = DRAINAGE MATTRESS.



TYPICAL CROSS SECTION OF USK DAM

FIG. 6·8


SEISMIC REGIONALISATION MAP
ARE INCLUDED IN PREPARATION OF THIS MAP
SCALE - KILOMETRES
100 0 100 200 400 500
DESCRIPTIONS OF THE ZONES
EQUIVALENT EARTHQUAKE INTENSITY MODIFIED MERCALLI SCALE
ZONE 4 DAMAGE XI
ZONE 3 HEAVY DAMAGE IX AND X
ZONE 2 MODERATE DAMAGE VII AND VIII
ZONE 1 MINOR DAMAGE VI
ZONE 0 NO DAMAGE BELOW VI
SHOCKS IN THE PENINSULAR SHIELD
SRINAGAR
ZONE 3
DELHI
ZONE 2
KATMANDU
SHILLONG
ZONE 4
AJMER
ZONE 1
PATNA
ZONE 3
ZONE 2
ZONE 3
ZONE 4
AHMADABAD
CULCUTTA
NAGPUR
CUTTACK
BOMBAY
ZONE 0
HYDERABAD
BANGALORE
MADRAS
ZONE 3
36°
32°
28°
24°
20°
16°
12°
8°
4°
64°
72°
76°
80°
84°
88°
92°
96°
104°

in that the drainage path of individual season work was cut short to help early dissipation of pore pressure. Ukai Dam, 110' high, was constructed in 3 seasons, hence this intermediate drainage blanket could be conveniently provided at end of every season's work. In case of still higher dams, where the construction programme is such as to complete a lift of 50' - 100' in one season further alternatives will have to be thought over. For example, intermediate drainage blanket, say, sand filter at every 15' - 20' interval may be a good proposition. At Tenughat Dam (Bihar) where this problem is actually to be encountered next season, a similar recourse will be helpful.

Maslov(72) has given some Russian Practices to tackle problems of High Earth Dams density and conditions of seismic activity. Though he describes it, in some different context, author feels that the methods proposed for achieving stability of high earth dams constructed with hydraulic fill method, in seismic zone can be well adopted in our country with rolled fill dams also. Russians prefer to use sand in dam construction because of its higher shear strength and involving less drainage problems. N.N. Yakolov, an U.N. Expert at Central Water & Power Commission has also been advocating very strongly about using sand material as far as possible on high earth dams in India. Because of its having good draining properties and shearing strength, sand can be preferred for such construction.

Some sections with sand as major construction material can be proposed as shown in Fig.6.6 within limits of economy for water conservation. Any problem pertaining to its stability under static or dynamic condition (e.g. Earthquakes) can be dealt with at Design stage, supplemented by laboratory tests. But for the

Construction Engineers, above alternatives are reasonable and can be well adopted in India or other countries because we can accelerate the progress of wet construction without fear of endangering the stability of earth dam section.

A third alternative can be suggested to reduce the thickness of impervious section of earth dams to minimum, which is a potential source of danger to wet construction on earth dams and likely to cause high pore pressures.

6.3. PERMEABILITY CONTROL

Passage of water through or under an earth dam is known as percolation, its measure is called permeability and the factor relating permeability to unit conditions of control is termed as coefficient of permeability. Since, aim of a construction engineer amongst others is to allow as little loss of water through or under an earth dam as possible (which is possible by making impervious core impermeable) control of permeability is to be studied in this context.

6.3.1. RANGES OF PERMEABILITY

For purposes of earth dam construction, coefficient of permeability for impervious, semipervious or pervious materials have been accepted as below:-

Soil	Coefficient of Permeability
Impervious soil	1 foot a year
Semipervious soil	1-100 ft per year
Pervious soil	Greater than 100 ft/year

However, depending upon availability of materials locally even the pervious soils (usable as shell material) have been used earlier as impervious parts of earth dam (core material) and vice versa. In differentiating between soils for different parts of a dam, a ratio of permeability of 100 is generally desirable but 10 is acceptable if a greater range is not readily obtainable.

6.3.2. PERMEABILITY CONTROL MEASURES

Control of water loss by way of percolation can be achieved in the following three ways:-

(1) By reducing coefficient of permeability of impervious material of the soil fill or foundation strata.

(2) By reduction of hydraulic gradient by increasing seepage path

(3) By control of the effluent or hydraulic head by providing filters in the dam and coring.

Of the three ways stated above, (2) & (3) are matters relating to Designs and would not be discussed here. The first point i.e. of reducing coefficient of permeability of impervious material of soil fill can be achieved at site by construction engineer in further three ways:-

(1) By judicious selection of impervious material and quality control

(2) By compaction control

(3) By the use of additives or grout

MATERIAL CONTROL

Once selection of impervious material is made for use in the body of earth dam, they should not be intermingled with soils of other borrow areas, earmarked for use in semipervious or pervious zones. This needs visual inspection as well as borrow area control, discussed in Chapter 3, earlier.

PERCOLATION CONTROL

Since, loss of water by way of percolation depends upon amongst other factors, the voids of soil, the lesser the voids, the lesser would be the loss of water through body of dam. Hence, compaction control should be effectively met with, which densifies the soil by reducing voids. Chapter 3 and 4 deal with achieving required compaction for the dam section by better quality control and choice of compaction methods. But it does not give idea of actual permeability of the compacted soil, which depends upon stratification of soil, besides the density achieved for the section. There are different methods to find out coefficient of permeability in the laboratory, but for knowing actual permeability of compacted soil, in-situ permeability tests should invariably be preferred.

Designation E 18 of U.S.B.R.(9) is a field test for knowing water loss through a bore hole in the foundation. This has certain limitations and is able to give only approximate results. For example, it can give integrated permeability of a strata without acquainting whether the strata is stratified or homogeneous. Knowledge of the latter part is of as much importance because, it will help taking measures to scrap and re-roll earthfill or search for some alternative impervious material. Again there is mathematical limitations of the formulae used for field permeability tests. U.S.B.R. does not include these tests for construction control of earth dams, and control permeability by controlling compaction efforts and quality control. Author feels it as indirect way of exercising permeability control and does not think it a correct practice. In the absence of any new in-situ permeability tests, it would be desirable to adopt U.S.B.R. in-situ permeability test Designation E 18 or E 19(9) for frequent checks, say once in every 10 ft lift of earth fill, the length of dam being restricted by convenient working in one stretch say 1000 ft or more. Some Swedish Methods for Field Permeability control can be seen at Appendix 3.1 of Chapter - 3.

USE OF ADDITIVES OR GROUT

Chemical grout or clay grout or a process which helps sealing or reducing the voids of the compacted fill can reduce permeability of the soil to a certain extent. Similarly, use of additives (say soil-cement) may be helpful in giving a less pervious membrane for the earth dams, where impervious soils are not at all available, but their efficacies remain yet to be established, because such techniques have not yet been adopted for high earth dams. However, use of additives or grouts have been successfully used for improving the permeability of materials in case of backfill material of cutoffs. Both Obra and Tenughat Projects have used grouting to compact and densify sand between two concrete diaphragms. Use of additives in the body of the dam can as well be taken up cautiously though there may be chances of developing some extra pore pressure which can be overcome by providing adequate drainage arrangements during construction.

CHAPTER - 7

CONCLUDING REMARKS AND A CRITICAL REVIEW OF CONSTRUCTION CONTROL AT TENUGHAT DAM

7.1. CONCLUDING REMARKS

Construction Control of High Earth Dams can be exercised in two ways:-

(1) By controlling quality of the works

(2) By controlling progress of the works

7.1.1. QUALITY CONTROL

To control quality of the works requires certain specifications to be framed and strictly followed. Such specifications are of two types:-

(a) Performance Type

(b) Quality Type

Both types are equally good and useful in achieving desired quality in the works provided only correct specifications are adopted and followed. Laboratory and Field Investigations are helpful in framing such specifications. Such investigations are also useful to study problems peculiar to a particular site and thereafter to find out their remedial measures. Foundation treatment need be applied very correctly and, adequately after studying site problems.

7.1.2. TIME CONTROL

To control progress of the works requires that time at disposal should be utilised to the fullest, by manipulation of all the resources at command. Critical path method (network technique) is the best tool for it and should be applied to all projects big or small. To reduce total time of the project without loss of quality, progress has to be accelerated with methods which reduce time of individual activities but do not dispense with quality at the same time. For example, passing a fill layer depends upon how early moisture content and density of the fill can be determined. Rigorous methods take too much time while approximate method cannot be relied too far. There are still some rapid methods e.g. Hilf's Method, K.L. Rao's Method which give enough assurance of the results being adequately correct for field purposes. Some ultra modern methods based on nuclear and Refractive Index Principles have also their great future in progress control after such apparatuses are manufactured on commercial scale. Penetration tests for assuming moisture content and density of soil fill are also useful for rapid construction control. Some of the important factors which help in accelerating the progress are summarised below.

(i) Adopt the quickest feasible methods of determining field density moisture content and permeability coefficient

(ii) Fix "Compaction Ratio" to a minimum possible (say 90 %) which would help accelerating progress but without sacrificing design value of density and moisture content of the compacted fill

(iii) Decrease sampling frequency i.e. rate of frequency depending on uniformity of materials and field environments.

(iv) Increase thickness of layer of earth by using improved methods of compaction units befitting the type of soil available in borrow area.

(v) Decrease number of passes of rollers by using heavy compaction units without sacrificing any design criteria

(vi) Exercise rigid control of borrow area for avoiding undesirable materials to come to embankment

(vii) Frame correct specifications based on test sections

(viii) Use C.P.M. (Network Technique) to coordinate between different works and resources at command.

(ix) Employ instrumentation like piezometer and settlement cross arms to ensure developing pore pressures and settlements within limits both during and after construction of the dam.

7.2. <u>CONSTRUCTION CONTROL AT TENUGHAT DAM - A CRITICAL REVIEW</u>

<u>GENERAL</u>

Tenughat Dam with maximum height of 148 ft is rolled, zoned earth dam, in Hazaribagh District of Bihar and covers a total length of about 4 miles (312 Ch) including dykes and embankments. The dam falls in a region where average maximum rainfall is over 58" and is located over River Damodar, draining through two Districts of Bihar, Palamau & Hazaribagh before it enters into West Bengal State. Panchet Hill Dam (D.V.C.) is about 80 miles D/S of Tenughat Dam site. Main aim of the project is to cater to the needs of Bokaro Steel Plant which is located about 25 miles D/S of the dam site and to feed other ancillary industries to come up in the region. The site is located in a hilly tract of Chotanagpur Division and valley is quite wide. Availability of soil and sand as construction material in abundance at dam site, and the valley being wide, almost dictated the choice of an earth dam. A general alignment of the dam is shown in enclosed drawing (Fig.7.1). The dam alignment between 0 Ch to 100 Ch. is not in straight with that between 100 to 312 Ch. because of topographical reasons viz. ends have been connected to available hills on the alignment. Rock strata along dam alignment is shown in Fig.7.2.

7.2.1. <u>FIELD INVESTIGATIONS</u>

(a) <u>CONSTRUCTION MATERIAL</u>

To ensure enough quantity of construction material being available at the site, detailed field investigations were taken up, before starting actual construction. Borrow area investigations for

TENUGHAT DAM PROJECT.

TENUGHAT DAM ~~PROJECT~~ ALIGNMENT

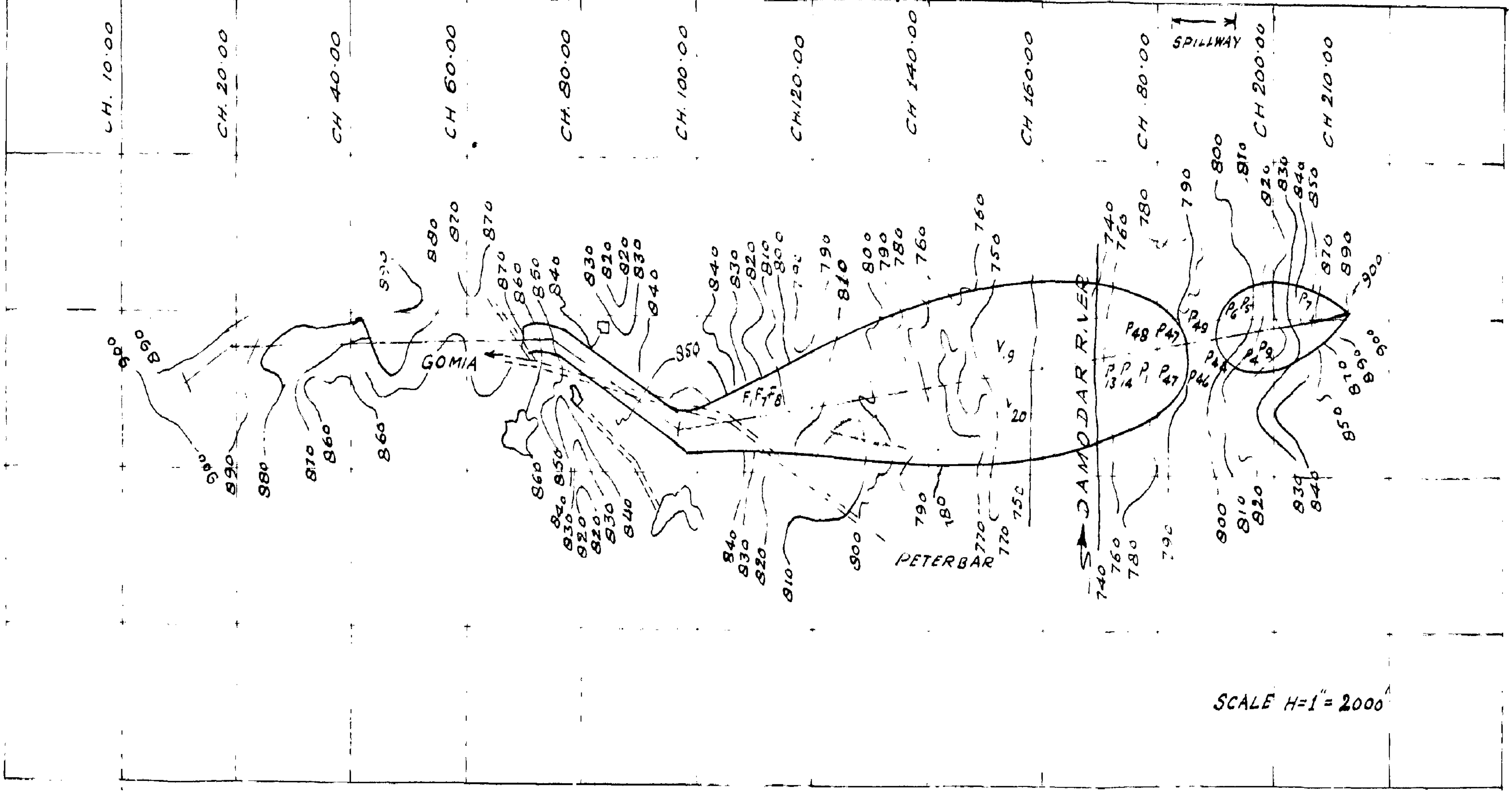


FIG. 7·1

TENUGHAT DAM PROJECT

PROFILE OF SOIL AND ROCK STRATA ALONG CUT-OFF TRENCH.

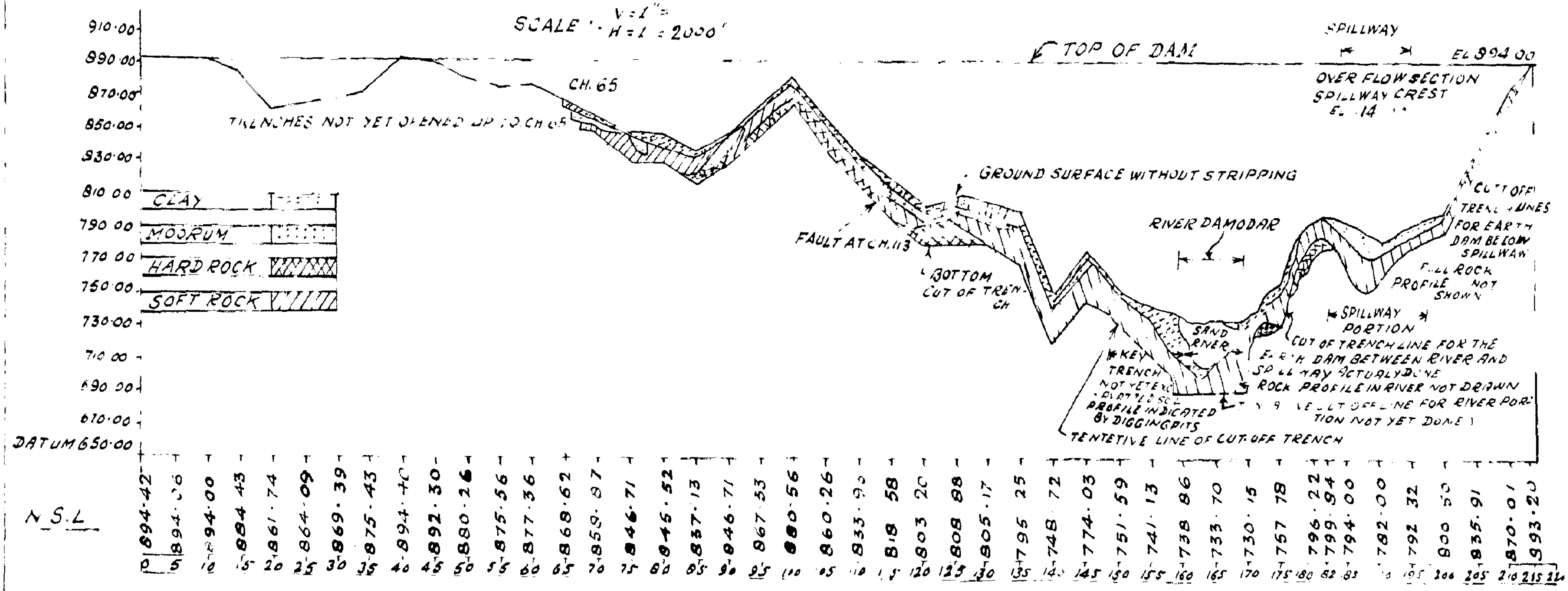


FIG. 72

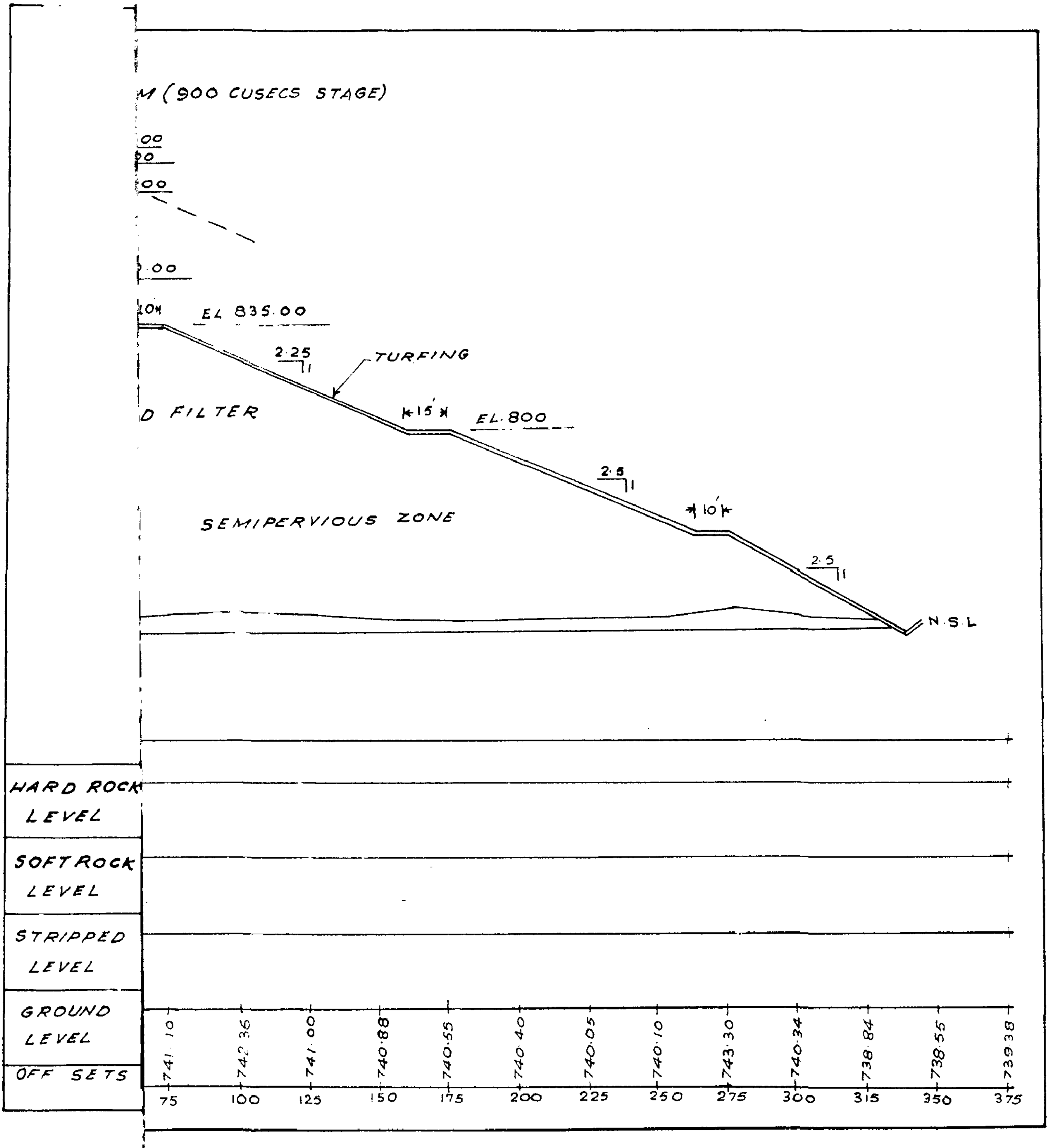
impervious and semipervious materials revealed that though soils of varying densities between 112 to 116 lb/cft were available at site in abundance, the permeability were less than 10 ft/year which is usually classified as semi-impervious soil. Hence it could be concluded that though there was no dirth of casing material at site materials for impervious core of the earth dam as available at site was not very encouraging. Good river sand as available in the river bed was also in limited quantity and not enough to meet the full requirements if it were used in the "casing" of the dam. On the other hand, River sand was found to be satisfying filter criteria of sand filter in case soils having density 116 lb/cft (available in abundance at site) were used in the casing. Accordingly, the original provision of graded filter in the earth dam section was replaced by sand gravel and it was decided to use the above soil as casing material with precautions taken to improve drainage, both during and after construction. Use of sand filter instead of graded filter brought about a considerable saving, because good quarries were located very far off. To improve drainage, intermediate drainage blanket of sand (3' thick) @ 15 ft intervals at U/S and, an effective sand chimney at D/S were provided. The former sand drain on U/S face will improve drainage through casing material of dam both during construction and in case of sudden drawdown conditions after construction of dam. Effective sand chimney on D/S of core will keep down the phreatic line during steady seepage when the reservoir is filled and it is expected even during the period of construction, it will provide for adequate pore pressure relief for the casing materials being used. On flanks no piezometers have been so far installed in the body of the dam, hence their behaviour how construction pore pressure behaved could not be studied. But next season (1967-68) when works in river bed will be taken up on earth dams, adequate piezometers will be provided for studying pore pressure dissipation during and after construction. On flanks, however, placement moisture content was tried to keep on dry side of optimum to control the rise of construction pore pressure. Scarcity of impervious clay as pointed out earlier was overcome by making section of the core thin and sloping. (Vide Fig.7.3)

(b) SOIL COMPACTION

Laboratory investigations earlier carried out at Central Research Station of the Project found that maximum dry density of core and shell materials, as 118 lb/cft and 110 lb/cft could be obtained at 16 % and 13 % O.M.C. (Proctor's Test). To study the behaviour of soil compaction at site, two test sections of 800' each were taken, and core and casing materials separately compacted over different plots. Excavations at borrow area had to be carried out by scrapers, because only 8' - 10' overburden could be used as core and casing materials and shovels could not be economically operated to work without making a face. Borrow area material were tested in laboratory and found to contain about 3.8 % short of that of optimum. Economics of watering the soil at borrow area and embankment were studied and it was decided that watering of soil at embankment would be economical, because River is situated almost midway of the dam length and pumping of water from River to dam area will involve less lead than that to the borrow area quite scattered, on the U/S of the dam axis. Besides, flooding of borrow area would have proved a handicap in works because of the working by scrapers.

So, soils laid at test sections, watered by water tankers in different proportions ploughed with harrows and compacted by 10 ton sheep foot rollers in layers of 9" thickness. Eight passes of sheep foot rollers, after watering of soil spread over 800' x 250' area, was ultimately found to give 98 % compaction at 1 % dry of

O.M.C. for eith r of core and casing mate ials. Further rolling did not give any marked improvement in increasing the density. An



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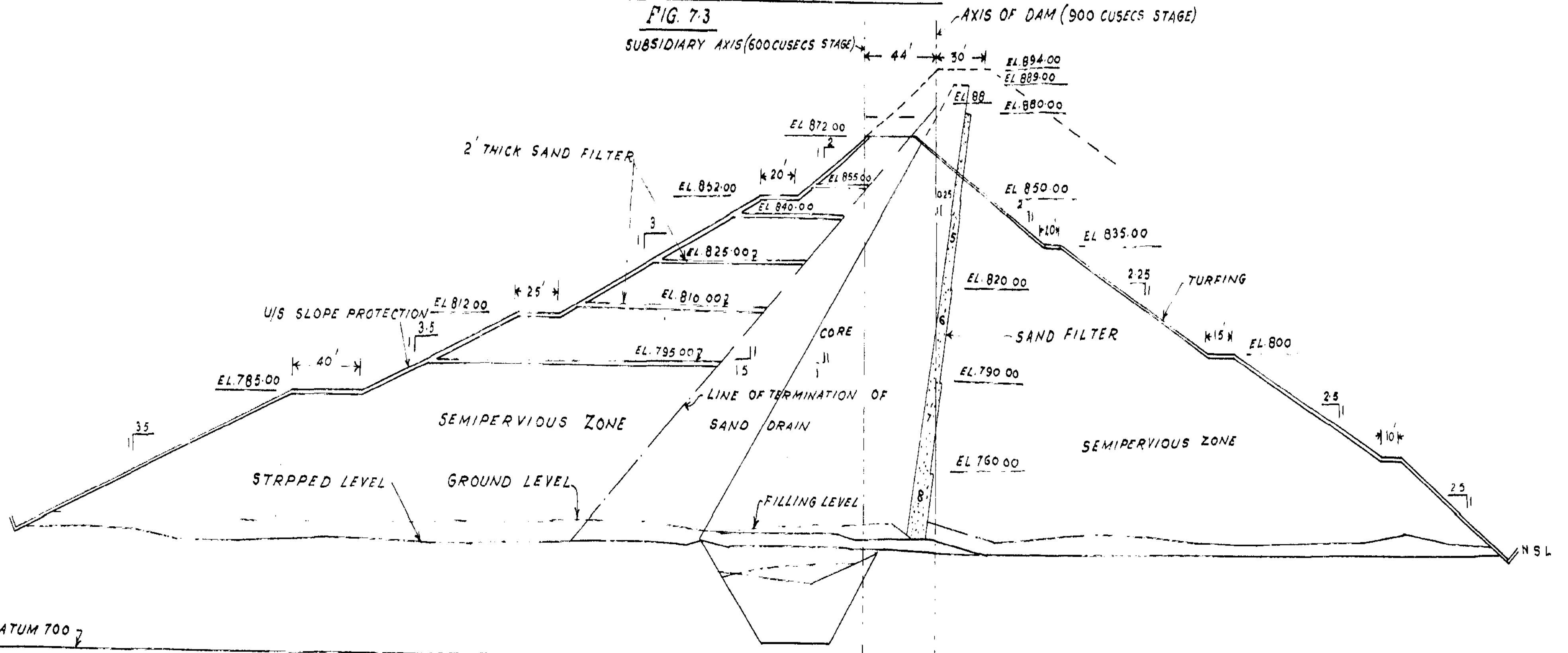
weather but in any case the above maximum dry densities as specified above were to be ensured.

CROSS SECTION OF DAM AT CH 171

FIG. 7.3



DATUM 700

OFF SETS	GROUND LEVEL	STRIPPED LEVEL	SOFT ROCK LEVEL	HARD ROCK LEVEL
550	742.00	742.19		
525	745.31	741.72		
500	744.82	740.28		
475	743.87	740.45		
490	743.77	737.56		
425	743.72	738.43		
400	743.87	737.45		
375	743.37	339.83		
350	745.63	738.82		
325	744.57	738.17		
300	744.12	737.76		
275	743.67	737.92		
250	743.77	738.23		
225	745.87	739.27		
200	745.37	739.27		
175	744.61	738.83		
150	743.40	737.42		
139	742.00	740.00		
30			738.00	729.25
25	741.02	729.50	737.50	729.00
00	741.04	726.68	738.00	730.00
75	741.30	726.86	738.21	732.00
50	739.64	732.56	739.42	732.56
25	739.63	739.05	739.00	737.75
0	739.72	739.00		
25	739.68			
50	739.96			
75	741.10			
00	[illegible]			
25	741.00			
50	740.88			
175	740.55			
200	740.40			
225	740.05			
250	740.10			
275	743.30			
300	740.34			
315	738.84			
350	738.55			
375	739.38			

O.M.C. for either of core and casing materials. Further rolling did not give any marked improvement in increasing the density. An increase in water content to over 1 % wet of O.M.C. was not found useful as it presented working difficulties with sheep foot rollers. Since scrapers were utilised for excavating and moving the earth it did not require much of clod breaking. However, use of a plough was considered adequate.

Hence, following decisions were taken and later followed for soil compaction.

(a) COMPACTION METHOD

Compaction by 10 tons sheep foot roller both at core and shell sections of the earth dam.

(b) PLACEMENT MOISTURE CONTENT

1 % dry of the optimum for both of impervious and semipervious material as below:-

	Maximum Dry Density		Moisture Content	
	Lab. Value	Field Value	Lab O.M.C.	Field Placement
Impervious	112 lb/cft	100.8 lb/cft	16 %	10 %
Semipervious	116 "	113 "	12 %	11 %

It was also decided to allow 1 % wet construction, in certain cases, if the works were met with rains during fair weather but in any case the above maximum dry densities as specified above were to be ensured.

(c) FIELD PLACEMENT CONTROL

Soil to be compacted in 9" layers, free of woods and organic matters(to be picked up at embankment). No tolerance to be given for this, as practically no such materials were met with in borrow area. Eight passes of sheep foot rollers would ordinarily give required soil compaction. But for passing fill layers, quality type specifications will be adopted i.e. by ensuring required maximum dry density and placement moisture content as in Cl.(b) above. At Tenughat Dam, passing a fill layer is still continued by comparing the wet density of the compacted fill with that of the field values of wet density (calculated at 10 % or 11 % placement moisture content) and moisture content is determined later on with oven drying method. This is not a correct procedure in view of the construction of a high earth dam. However, efforts are being made to make use of Hilf's Methods of Rapid Construction Control from the next working season when construction speed is also supposed to go high. Use of Rao's method will be more appreciated as this will be quicker than the Hilf's still giving comparable results.

7.2.2. PORE WATER PRESSURE IN IMPERVIOUS AND PERVIOUS FILLS

No piezometer installation have been made so far, where the works were continuing on the flanks of the river. But elaborate drainage arrangements both U/S and D/S of core have been provided for quick dissipation of pore pressure. Also because, on flanks earth dam construction was spread over three seasons, no abnormal rise in construction pore pressure were anticipated. This year however, since 142' high earth dam in a length of 1000 ft is to be completed in one working season of 8 months, speed of construction warrants providing enough of drainage arrangements and also recording of construction pore water pressure at various heights of construction. Providing horizontal pervious filters at 10' interval on D/S, similar to that on U/S is expected to help dissipation of pore pressure relatively quickly. Piezometer should also be installed in pervious fills and filters to study their efficacies.

7.2.3. PORE WATER PRESSURE IN PERVIOUS FOUNDATION DUE TO SEISMIC ACTION.

The region in which Tenughat lies has experienced earthquakes of maximum intensity of VI in the past. Hence, it was desired to simulate such seismic condition and ascertain whether pore pressure is likely to rise during earthquakes to such dangerous proportions as to cause liquefaction of river sand. Method and results have been fully discussed in Article 6.1 and from that it was established that for design earthquakes of intensities VI, liquefaction of River Damodar sand was a distant possibility.

7.2.4. FIELD PERMEABILITY TESTS

These tests are necessary to decide depth of cutoff trench and foundation treatment as well as for confirming definite ratio of coefficient of permeability in different zones of a zoned fill embankment. At Tenughat Dam Project, these tests have been performed all along the alignment of the earth dam and foundation treatments have been provided accordingly. Field permeability tests in bore holes as specified by Designation E-18 of U.S.B.R.(9) were conveniently followed for such purposes while that for day to day work well permeameter method as specified by Designation E-19 of U.S.B.R.(9) were followed.

7.2.5. FOUNDATION TREATMENT

6 miles long alignment of Tenughat Dam brings in a [illegible] many construction problems but of which very important one are a tectonic

fault, artisan water, River Damodar of pervious bed and localised patches of top day strata crossing the alignement of the main dam. Besides, the underground strata have been found in certain patches to comprise of excessive permeability due to fractured schistose and biotite gniess . Following treatments have been done or are proposed to be completed to get over the problems.

TECTONIC FAULT

This is a boundary fault crossing dam alignement at Ch. 113.00 (Fig.7.2) composed of deeply weathered composite gneisses with bands of biotite and pegmatite between Ch. 113.47 - 117.87. The strata have been found to be soaked quite deep and excavation of the entire weathered rock and back filling with concrete or compacted clay was not found to be economically feasible. In such circumstances the cutoff of the earth dam has been proposed to be taken below overburden say 3 ft into hard rock and employ grouting below the cutoff to depths and pattern still being investigated and planned. Insitu maximum permeability between Ch. 113.47 - 113.87, affected by fault has been found to be 50 litres (per minute per 5 ft pressure of water per square inch of area) at a depth of 60 ft. This indicates what a tremendous loss of water would result if the area went untreated.

ARTISAN WATER

Artisan water coming through the hole at Ch. 148.00 gives clear indication that the underground water is under pressure. Sometimes, water pressures has been observed to reach 2.5' above R.L. 746 (stripped ground level). Since this zone crosses the alignment of the dam, internal drains leading to outfall drains will have to be installed with adequate filters provisions to drain off the artisan water and help relieving pore pressure after the dam is built.

RIVER DAMODAR

Since River bed comprises of maximum 60' deep sand over deeply weathered gneiss and schistose, economics of open trench cutoff and concrete diaphragm by Rodio Marconi Method were studied and the latter has been found to be economical and adopted and completed (Chapter - 5) (Vide Fig.7.4)

IMPERVIOUS STRATA OVERLYING PERVIOUS STRATA

Such thick strata close to the present river bed on left bank, unless treated adequately, is apprehended to cause a case of ~~slight~~ slide failure at the junction of impervious top layer (clay) with the underlying pervious layer (moorum). Besides, development of excessive uplift pressure below downstream foundation is also anticipated because of stratified foundation. Recourse to pressure relief wells on downstream will be an obvious solution in the latter case; but the former problem still remains unsolved. One of the proposal is to employ encasing of impervious such strata within foundation between two concrete cutoffs. This will ensure a safety against sliding at the junction.

Where such strata is of a smaller depth, it will be entirely excavated and backfilled with pervious material below the casing of earth dam. Elsewhere where entire excavation of such strata would not be effected due to economical reasons, adequate pressure relief wells will have to be provided after construction of dam.

GROUTING

Tenughat Project Dam being mainly a water supply project every efforts have to be made to reduce water losses from various sources. Apart from the structural stability point of view, curtain

TENUGHAT DAM PROJECT (DETAILS OF CONCRETE CUTOFF)

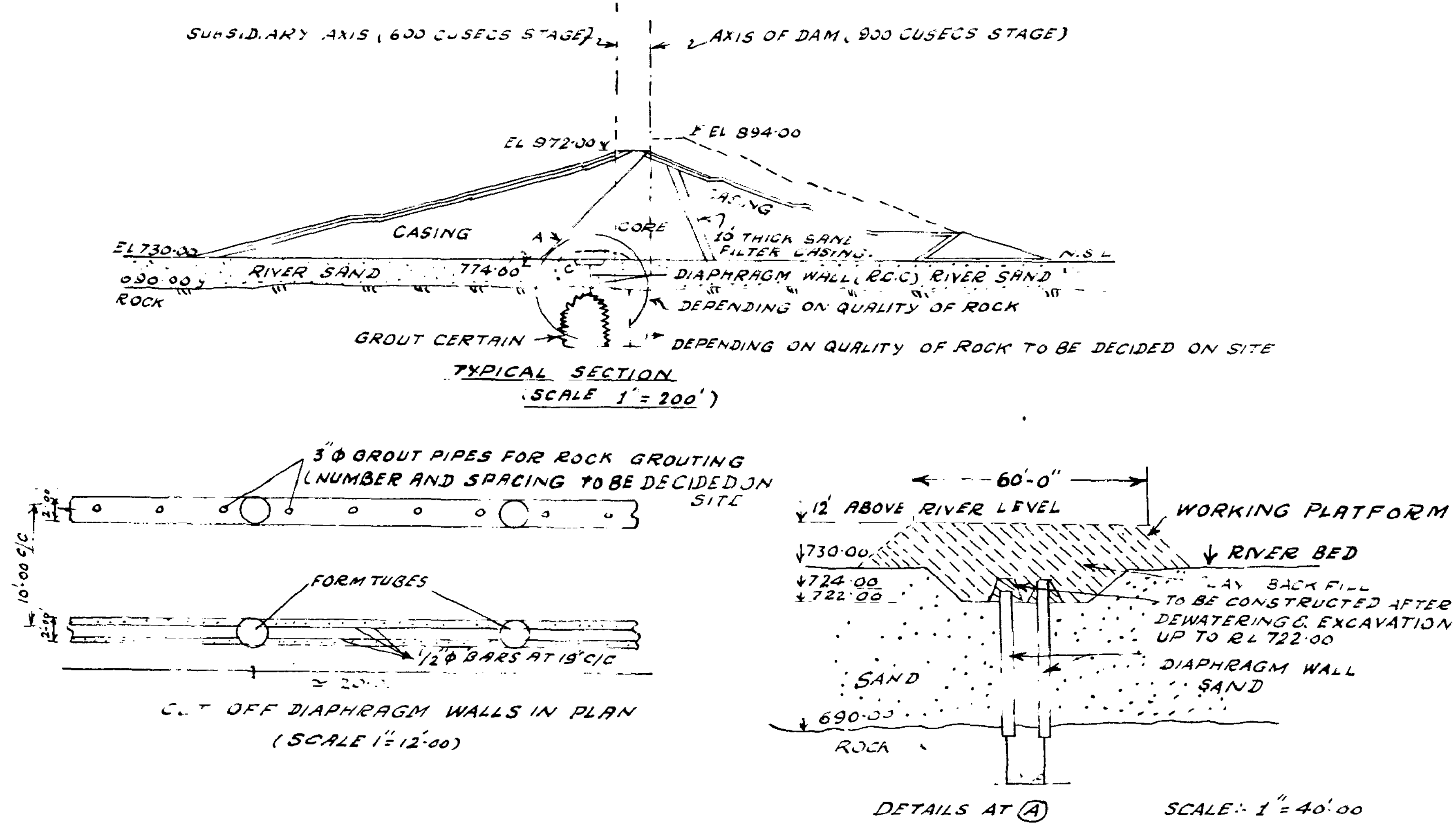


FIG. 7·4

grouting will reduce loss of water due to underground seepage.

ABUTMENTS

The extreme abutments of the earth dam are stable hills or high ground consisting of schistose and biotite gneiss. However to ensure proper bond between the earth dam and hills, stripping by light blasting will be done and good compaction assured at junction.

GENERAL FOUNDATION TREATMENT

Where the overburden was less say upto 25' (maximum) whole of it were excavated by scrapers and backfilled with compacted clay after thorough cleaning of the surface with water and air jets. Where required underground seepage water had to be removed by effective dewatering before laying and compacting the fill layers. In case where the overburden exceeded 25' depth, 2' stripping of top overburden was completed by scraper and dam constructed, followed by curtain grouting. After construction grouting (curtain grouting) facilitates reducing time schedule of the project.

7.2.6. INSTRUMENTATION

Adequate instrumentation has been proposed in the body of the dam, to observe settlement and pore pressure variations during and after construction of the dam. Their locations are given below:-

	Berm Level	Chainage
(a) Settlement Points	812.00 872.00 804.00	At odd chainages starting from 137+00 170+00 129+00
	852.00 834.00	At even chainages starting from Ch. 114+00 116+00
(b) Piezometer Installation	At varying elevation	120+10 148+90 164+90

SLOPE INDICATOR

One slope indicator tube (3.5" dia) has been embedded in the concrete diaphragm (Panel L-7) to record deflection of the same under lateral loadings, both before and after filling of reservoir. Slope indicator is comparatively a new device adopted in the Project to measure angular displacement of concrete diaphragm (in the River bed section) under lateral loadings. A brief description of the apparatus which is based on finding null point in a balanced electrical circuit, is given at Appendix 7.1 together with a set of observations taken on 21.7.67 to record initial deflection of the diaphragm from vertical plumb.

7.3.7. CONSTRUCTION SCHEDULE

Construction schedule of Tenughat dam has been prepared both by Bar Chart (Gantt Chart) and C.P.M. (Network Technique). A compaction study (Chapter -2) shows ~~HOW~~ that full control can be exercised over the progress of works only by the latter method besides its being helpful in reducing total time, of the project.

APPENDIX - 7.1

SLOPE INDICATOR

(Available from Slope Indicator Co. Seattle, Washington) (U.S.A.)

GENERAL DESCRIPTION

Slope Indicator measures slide movements, displacements within earth dams, bending of sheet piles bulk heads and other types of lateral earth movements. Instrument is water proof.

The instrument is normally lowered down on extruded alluminium or grooved plastic casing, the grooves controlling the orientation of the instrument in a predetermined direction. Inclination readings are taken at frequent intervals of depth and are subsequently converted to displacements. Consecutive readings at the same depth intervals at periodic intervals of time are used to determine depth and rate of ground movement.

The special grooved casing, plastic or alluminium can be installed in 4.5 inch or larger borings, or it may be built up with the embankments of earth dams or even epoxied to the face of sheet piling or beams. The plastic casing is available in five feet lengths and the alluminium casing is available in either 5 ft or 10 ft lengths.

Sensitivity = $\frac{1}{1000}$ i.e. a tilt as little as 3 minutes can be detected

i.e. a lateral displacement of 1" in 100" depth can be noted

Actually, Displacements in thin shear zones of less than 1/16 inch has been detected; earlier.

The instrument is so designed that the inclination by the plane defined by the four guide wheels is directly proportional to the potentiometer dial reading when the circuit is in balance.

DATA.

The slope indicator measures the inclination of the casing in a observation well at frequent intervals of depth. One set of readings is taken in one groove and an additional set is taken in the groove on the opposite wall of the casing. The instrument is adjusted so that the inclination of the casing from the vertical may be ~~compacted~~ computed by the following equation

$$\tan i = \frac{D_N - D_S}{2K} = \cancel{x000}k$$

Where D_N = Dial reading in Northernly groove

D_S = Dial reading in Southernly groove

K = 1000 for series 200 instrument

= 2000 for series 200-B instrument

= 3600 for series 100 instrument

APPENDIX 7.1 -(Contd)-

SLOPE INDICATOR READINGS

(TENUGHAT DAM PROJECT)

THE SLOPE INDICATOR CO.
FIELD SHEET
SLOPE INDICATOR DATA
TENUGHAT DAM PROJECT (BIHAR)
Left Bank Ch. 158+96.5 Panel L-7
Date - 21.7.67 INSTR - 200-B (Hence, value of K = 2000)

OBSERVATION WELL NO.1
Top El. of Observation Casing 743.3
Sheet - 1 of FIRST READING

S.No	Depth in ft	Diff	A_z = E +	S -		Diff	Change	Diff	A_z = E +	W -		Diff	Ch Dg
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(1
1	42.5		596	560	156	+36			606	551	157	+55	
2	41.0		599	556	155	+43			609	549	158	+60	
3	39.5		603	554	157	+49			609	546	155	+63	
4	37.0		606	551	157	+55			620	533	153	+87	
5	35.5		605	552	157	+53			622	534	156	+88	
6	34.0		603	552	157	+51			621	534	155	+87	
7	31.5		608	547	155	+61			609	547	156	+62	
8	30.0		608	548	156	+60			605	549	154	+56	
9	28.5		608	546	154	+62			603	551	154	+52	
10	26.0		615	540	155	+75			585	570	155	+15	
11	24.5		614	541	155	+73			585	.570	155	+15	
12	23.0		613	542	156	+71			587	569	156	+18	
13	20.5		630	526	156	+104			556	600	156	-44	
14	19.0		632	526	158	+106			553	601	154	-48	
15	17.5		629	527	156	+102			553	604	157	-51	
16	15.0		604	551	155	+53			566	592	158	-26	
17	13.5		603	552	155	+51			564	593	157	-29	

APPENDIX 7.1 -(Contd)-

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(1
18	[illegible]		602	554		+48			563	593		-30	
					156						156		
19	9.5		580	576		+04			574	582		-08	
					156						156		
20	8.0		581	574		+07			574	581		-07	
					155						155		
21	6.5		581	574		+07			576	581		-05	
					155						157		
22	4.0		563	593		-30			593	561		+32	
					156						154		
23	2.5		564	593		-29			595	560		+35	
					157						158		
24	1.25		565	593		-28			599	558		+41	
					156						157		

Note:- Dial readings indicated under column 'N' have been taken when fixed wheels of the slope indicator were pointing towards north.

CONCLUSIONS

(i) At depth 42.5 ft, $\tan i = \dfrac{D_N - D_S}{2K} = \dfrac{596 - 560}{2 \times 2000}$

$$= \frac{36}{4000} = \frac{9}{1000} = 0.009$$

Hence $i = 0^\circ\ 30'$ This gives initial position of the slope indicator (tube) fixed in U/S concrete diaphargm

(ii) At other depths, inclination of tube can be similarly calculated

(iii) Marked ~~variation~~ increase in worked out difference at 31.7, 13, 19, 22 indicate joints, where tube pieces have been coupled together, not being in one plumb, apparently.

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