# WORK DONE BY THE COMMITTEE ON CONSTRUCTION OF HIGH DAMS IN SEISMIC ZONES

*(Revised and Enlarged Second Edition)*

**CENTRAL WATER & POWER COMMISSION**
**Ministry of Irrigation & Power**

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सत्यमेव जयते

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## FOREWORD

The Ganga and Brahmaputra River Commission at its Seventh meeting held on 12th April, 1958 appointed a committee to examine the feasibility of constructing high dams, both masonry and earth, in seismic zones. This was a very timely and appropriate step in view of the fact that a number of dams have to be constructed on the rivers originating from the Himalayan ranges, not only to harness their waters for irrigation and generation of hydro-electric power, but also to control the recurring damage caused by the floods. As the potential risk involved in the damage to the dam structure due to earthquakes may be considerable, any rational design has to cater for the correct appreciation of the seismic forces. This has to be preceded by a good deal of laboratory and field experiments.

The present publication covers the work done by the committee so far; and I hope that this will provide very valuable material for getting an insight into the nature of the seismic forces and their effects on dam structures. As very little work has earlier been done in India on this subject, I am sure this will be a welcome addition to the engineering literature in India and as such it will get a good response from the engineers engaged in this field.

M. R. CHOPRA
CHAIRMAN
C.W. & P.C.



NEW DELHI
1963

सत्यमेव जयते

## PREFACE

Since the commencement of the planned era of National Development in 1951, as many as 57 large dams have either been built or are under construction for utilization of water resources for irrigation, hydro-electric power and flood control. The demand for water utilization is ever-increasing due to the all-round development of the country's economy, so more and more dams have to be built.

The upper catchments of the two mighty rivers—Ganga and the Brahmaputra—including their tributaries, where suitable dam sites can be located lie in seismic zones of varying intensities. It was, therefore, appropriate that Indian engineers started thinking on rational design of high dams to resist seismic forces. This Committee's work is a pioneering effort in that direction. Great interest has been evinced in bringing out the first edition of the publication summarising the work done by the Committee so far. This first edition got exhausted in a very short period.

It has, therefore, been decided to bring out the second edition. This includes further work done by the Committee since the first publication, short chapters giving the procedure of earthquake resistant design of dams in India and abroad, a comprehensive bibliography, as well as a map of India showing various seismic zones. It is hoped that this revised edition will be quite useful to the engineers engaged in this field.



After the completion of field and laboratory experiments, which the Committee has proposed to undertake, it would be able to give specific recommendations, which will be contained in its final report. This will not be the end, but rather a beginning for further sustained research and experimentation in this field yielding better designs of dams to cater for the seismic forces which hitherto somewhat restricted our efforts of building high dams in seismic zones. This is a challenge to the ingenuity of our engineers and I am sure, they will meet it in their best tradition.

I take this opportunity to place on record my deep appreciation of the good part played in this Committee's work and the assistance rendered in bringing out this publication by Sarvashri Y. K. Murthy, Director, Dams, B. R. Chopra and R. B. Shah, Deputy Directors, Central Water & Power Commission. Their efforts have been willingly made, despite the very pressing load of design work, which they have to carry.

29-8-1963

Sd/- (C. L. Handa)
*Member (Designs & Research)*
*and*
*Chairman of the Committee*

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# I. BACKGROUND NOTE

At the fourth meeting of the Brahmaputra River Commission held on 7th of March. 1956 at New Delhi, it was decided to constitute a committee to examine the advisability of constructing high dams in seismic zones. It was also decided that the Committee should examine only those dam sites which were under investigations in Assam and West Bengal.

The Committee consisted of :—

1. Shri Balwant Singh Nag, I.S.E., Chief Engineer and Secretary, Government of Assam — *Chairman*
2. Shri George Oomen, Director, Dam Designs, C.W.&P.Commission, New Delhi — *Member*
3. Shri P. C. Hazra, Superintending Geologist, Geological Survey of India, Calcutta — *Member*
4. Shri A. N. Tandon, Seismologist, Indian Meteorological Department, New Delhi — *Member*
5. Shri O. P. Mittal, Superintending Engineer, N.E.F. Agency, Shillong — *Member*
6. Shri D. P. Chatterjee, Superintending Engineer, North Bengal Circle, Jalpaiguri — *Member*
7. Shri Kehar Singh, Superintending Engineer, Assam Investigation Circle, Gauhati. — *Member-Secretary*

The Committee took the view that its deliberation should be confined to the consideration of advisability or otherwise of building dams purely from the considerations of the effects of earthquakes which include rapid siltation of reservoirs. Other factors such as the economics of building any particular dam or the difficulties of construction, were not to be considered.

"The Committee held three meetings and published its report. The report after giving the geological history of Assam and West Bengal including tectonics, describes the seismicity of the region along with a review of the major earthquakes with their epicentres and effects. It also contains a brief discussion of the effects of earthquakes on river systems, earthquake magnitudes, intensities and the scales of measuring them. It also indicates the requirements of the seismological data for engineering problems. The report then considers in detail the feasibility of constructing high dams at individual dam sites in Assam and West Bengal in the light of the data and other information available and gives its recommendations. A summary of the report and recommendations may be seen at Appendix A."

This report was considered by the Ganga & Brahmaputra River Commission at its seventh meeting held at Varanasi on 12 April, 1958. The possibility of constructing high masonry or concrete dams only was examined in the report. The River Commission expressed that the entire problem had to be reviewed to study whether the particular dam sites examined by the Committee were safe for the construction of earth dams also. The Commission was of the view that earth dams as high as 450 ft. could be constructed in the seismic zones as shown by the experience of other countries. The need for re-examination of each site as regards suitability of foundations for construction of high earth dams was stressed. For this purpose the Commission reconstituted the Committee as follows:—

| | |
|---|---|
| 1. Member (D&R), C.W.&P.Commission | *Chairman* |
| 2. Chief Engineer, Assam | *Member* |
| 3. Special Engineer, Flood & Flood Control, West Bengal | *Member* |
| 4. Representative of Geological Survey of India | *Member* |
| 5. Dr. A. N. Tandon, Seismologist, India Meteorological Department | *Co-opted Member* |
| 6. Superintending Engineer, Assam Investigation Circle, C.W.&P.Commission, Gauhati. | *Member-Secretary* |



The following members were subsequently co-opted by the Committee :

1. Chief Engineer, Irrigation Department, North Bihar.
2. Chief Engineer, Kosi Project, Government of Bihar.
3. Dr. Jai Krishna, Head of Structural Engineering Division, University of Roorkee.
4. Shri Y. K. Murthy, Director (Dams), C.W. & P.C.
5. Shri I. P. Kapila, Director Beas Project.

## II. MEETINGS AND DECISIONS

The Committee has since held five meetings. The first three meetings were held at New Delhi on 14-4-1959, 25-11-1959 and 21-11-1960 respectively. The fourth meeting was held on 3-11-1961 while the fifth meeting was held on 18-1-1963. The Committee is still continuing its deliberations. The decisions taken by the Committee in its various meetings are given below; while the action taken on the implementation of these decisions is described under para III.

### (a) Decisions Taken in the First Meeting

1. The Chairman desired that, first of all information and particulars in respect of the earth dams already built in the seismic areas should be collected, with particular reference to allowances made for seismic factors. It was stated that such structures would have already been designed and built in countries like America, Japan and Italy. It was mentioned that Trinity Dam (540 ft. high) and Anderson Ranch Dam which are being built in California might be some examples. Since the State of California is considered to be in seismic zone, it was deemed that the earthquake design criteria adopted for these dams would be of considerable help. Along with this information it was enjoined that a general study of seismic zones in America, *viz.*, that of California, Mexico, Nevada and Arizona as possible from the existing literature may also be made. All the available literatures on the subject should, as far as possible, be collected and briefed for the members' perusal.

Dr. Tandon agreed to help in getting information and other data on the subject for Japan and America which might be available with him at Delhi or in Shillong. He also suggested that the proceedings of the Symposium of Earthquake Engineering held recently at Roorkee might be of interest to the members of the Committee. As it may take considerable time for the proceedings to be printed and published, it was felt that copies of the papers submitted at the symposium, including those from Japan might be obtained from Dr. Jai Krishna for the benefit of members.

2. The Chairman desired that model experiments might be conducted at Roorkee simulating the seismic effect on earth dams. These experiments may be carried out on a vibrating table having on top different types and sections of earth dams, dam sections with and without cores may be studied for their behaviour under earthquake condition of varying intensities. For conducting these experiments, the Seismological Laboratory recently started at Roorkee University was considered suitable. Dr. Jai Krishna, Associate Professor of Structural Engineering, Roorkee University, was requested to arrange for conducting these experiments. It was suggested that he might be invited to come to Delhi some time in May 1959 for discussion with the Chairman and Dr. Tandon regarding the nature and scope of these experiments.

It was agreed that the next meeting of the Committee should be held at New Delhi somewhere in August or September 1959. This would give adequate time for the collection of all the above mentioned data and to formulate further lines of action.

**(b) Decisions taken in the Second Meeting**

1. Collection of more information on earthquake resistant design of earth dams was considered necessary.

2. It was decided to address Dr. Susumu Nagata, Vice-President, International Commission on Large Dams and Mr. Carlo Semenza of Italy, requesting for literature regarding the practices followed in their countries for earthquake design of earth dams. They were also to be requested to give observations recorded regarding any failure of dams due to earthquakes and about the different experiments conducted in their country to derive the design criteria for earthquake effects on high earth dams.

3. The Committee decided to consider in detail the following earthfill and rockfill dams which fall into the earthquake zone and which are likely to be constructed in the near future :—

(i) Kopili Dam on Kopili river in Assam.
(ii) Barak Dam on Barak river in Assam.
(iii) Teesta Dam on Teesta river in West Bengal.
(iv) Noonthore Dam on Bagmati river in Bihar.
(v) Ramaganga Dam on Ramaganga river in U. P.
(vi) Pong Dam on Beas river in Punjab.
(vii) Kosi Dam at Kothar.

The following particulars were to be collected for the above seven dams for circulation among members :—

(a) Report of the Geologist on the dam site.
(b) A note on the topography.
(c) Rough section of the dam.
(d) Seismic characteristics of the area.
(e) General specifications.

Shri Hazra desired that the soils in the borrow areas of these dams should be analysed and their properties determined. It was decided that the nearby Research Institution should be asked to carry out the following tests on the soils at the dam site :—

(a) Mechanical analysis.
(b) Atterberg's limits.
(c) Shear strength.
(d) Permeability.

4. Chief Engineer, Assam, should be approached to move his Government for sending some intelligent Assam engineer abroad to U.S.A. and Japan for further study on the subject.

5. Regarding model experiments, it was decided that :—

(i) Dr. Jai Krishna should be associated for conducting these experiments.

(ii) The Council of Scientific & Industrial Research should be requested to equip their laboratory at Jorhat (Assam) with suitable apparatus to conduct all experiments on earthquake effects on structures and dams.

(iii) The C.W.&P.R.S. Laboratory should be equipped for carrying out model experiments.

6. Dr. Tandon said that seismic zoning maps are being prepared for India by a panel constituted by the Indian Standards Institution, of which he and Dr. Guha of C.W. & P.R.S. are members. He expected that they should be able to bring out these maps in about 6 months' time. It was decided that Dr. Guha should be contacted to make the maps available for the members of this Committee as soon as they are ready before the next meeting of the Committee, if possible.

**(c) Decisions taken in the Third Meeting**

1. It was decided that the scope of the Committee should be enlarged to cover all the river valley project structures, barrages, dams, etc. and not only earth dams.

2. The Chairman of the Committee desired that Member-Secretary should make a compilation of the work so far done in a bound volume, and a copy of the same may be sent to each of the members.

3. Dr. A. N. Tandon was requested to prepare a note on earthquake occurrence, features, etc. In this connection the Member-Secretary was to obtain and to supply to Dr. Tandon a list of the projects which are under construction as well as under investigation.

4. It was decided that Dr. Jai Krishna be requested to collect the data necessary in regard to :—

(i) Laboratory experiments.

(ii) Field experiments.

5. It was also decided that Shri Y. K. Murthy Superintending Engineer, Dams, Central Water & Power Commission and Shri I. P. Kapila may be requested to prepare a note in regard to :—

(a) The procedure of design of high dams in seismic region in India; and

(b) The procedure of design of high dams in seismic regions abroad.

6. Literature published by the Japan Society of Earthquake Engineering was decided to be collected for circulation among the members.

**(d) Decisions taken in the Fourth Meeting**

**(e) Decisions taken in the Fifth Meeting**

## III. IMPLEMENTATION OF THE DECISIONS

### (a) Action taken on the Decisions of First Meeting

As decided at the First Meeting of the Committee, available literature on the subject was collected and summaries of papers along with relevant extracts concerning "earth dam design in seismic zone" were circulated among the members. It has been found that literature on the earthquake effect on earth dams is rather meagre. The extracts from the following papers and publications were circulated :—

(i) Seismic stability of earth dams—by E. E. Esmiol, Technical Memorandum 641 of USBR (Appendix A).

(ii) Seismic considerations in the design of dams—by C. D. Mitra (Appendix B).

(iii) Design of dams for earthquake resistance—by J. M. Rapheal (Appendix C).

(iv) Special problems relating to the construction of dams in active volcanic country—by Hunter and Keefe (Appendix D).

(v) Measurements of settlements at certain dams on the TVA system and assumptions for earthquake loadings for dams in the TVA area—by Blee and Meyer (Appendix E).

(vi) Hydrodynamic pressures on dams due to horizontal earthquake effects—by C. N. Zangar, Engineering Monograph No. 11 of USBR (Appendix F).

Although completely satisfactory methods of evaluating seismic stability of earth dams have not been developed, a procedure for investigating the stability has been suggested in the USBR Memorandum 641 mentioned at item (i) above. It describes the adoption of the modified slip circle method taking into account the earthquake stresses. The condition of resonance, which causes most damage, has also been considered and the method of mitigating the same has been discussed. Other broad conclusions that can be drawn from the various papers are :—

1. Of all earthquake resistant dams, earthfill dams come first, next come rockfill dams and then the solid concrete gravity dams.

2. The shearing resistance of soil is greater under dynamic loading than under static loading. The cohesion of soil increases 1.5 to 2 times the original value for cohesive soils. Thus the increased earthuqake forces in the earth dam are somewhat balanced by the increased resistance of soil to stress and deformation.

3. The following special provisions are suggested to be made in the seismic design of earth dams :—

(a) Provide liberal free board to guard against overtopping in case of foundation settlements.

(b) Carry cut-off trench deep into foundation to intercept water flow through foundation cracks.

(c) Make the impervious section wider.

(d) Prevent slumping by weighting down impervious section with coarse pervious material.

(e) Provide extra heavy outer slopes to close any tension fissures that might be developed by stretching of the dam.

As promised in the First Meeting Dr. Tandon had brought in the Second Meeting the literature on America and Japan earthquakes. It was observed that these countries have drawn maps giving seismic zones. From these maps it is possible to get an idea of what earthquake acceleration can be expected in a particular region. He said that similar seismic zoning maps are being prepared for India by a panel constituted by Indian Standards Institution of which he and Dr. Guha are members. Accordingly Dr. Guha was requested to keep the Member-Secretary informed about the progress of this work and send copies of the maps (even if in the draft stage) before the next meeting for the benefit of the Committee. Dr. Guha has replied that the plotting of basic data required for the preparation of said maps has been completed and is under check and scrutiny. In order to have a proper idea of seismic zoning in different parts of India, these maps and other maps etc., are to be considered together and compared so as to have a reasonably correct division of India into seismic zones. These comparisons will be done at the next meeting of the panel of experts attached to the Earthquake Engineering Sub-Committee of Indian Standards Institution which was scheduled to meet at the end of the year 1960. The final seismic maps are, thus, expected to be ready after these comparisons of various maps have been made.

Further Dr. Jai Krishna had sent copies of papers read at the 1959 Symposium on Earthquake Engineering held at Roorkee. These papers were examined and one of them which concerned mainly with dam design was circulated to members.

As desired by the Committee in the first meeting, correspondence was made with USBR and Tokyo University of Japan regarding earthquake design of earth dams. Particular reference was made to the design of Trinity Dam in California. The reply from USBR has been received which has been circulated to members (Appendix G). USBR has adopted a design acceleration of 0.05 g both vertically and horizontally in the case of Trinity Dam.

As desired in the second meeting, letters were written to Japan and Italy by Dr. K. L. Rao, Member (D & R) and Chairman of the Committee for supplying literature on the subject of seismic zones. The reply has been received from Carlo Semenza from Rome. The information received from Mr. Visentini (Appendix H) has been circulated to all members and the papers which were in Italian language, have been translated into English and are enclosed as appendices K and L. No reply has been received from Japan.

Dr. Jai Krishna, Associate Professor of Structural Engineering, Roorkee University was addressed by the Chairman of the Committee to meet him for discussing the mode of conducting model experiments on the subject. The Vice-Chancellor of the University has permitted Dr. Jai Krishna to undertake these experiments at the University and to participate in the Committee meetings.

**(b) Action taken on the decisions of the Second Meeting**

As decided in the second meeting the following reports in regard to earthfill and rockfill dams, which are likely to be constructed in the near future, were collected and circulated among the members :—

(i) Report on the geological investigations of the Kopili Dam Site, Assam, November 1959—by P. N. Mehta.

(ii) A geological note on the cores recovered from Borehole No. 2-A, Barak Dam Site, Mainadhar, Cachar District, Assam (with 2 plates)—by A. Acharya.

(iii) Inspection note on Noonthore Dam Site on Bagmati river Nepal—by P. K. Chatterjee.

(iv) A geological report on the Ramaganga Project, Garhwal District, U.P.—by S. P. Natiyal, S. C. Avasthi and S. Bose.

(v) Progress report on the geological exploration on the Pong (Beas Dam Site, Punjab (a) with plates in a separate folder (February 1959)—by S. N. Chaturvedi.

(vi) A geological report on the proposed 400 ft. composite dam near Kothar village, Kosi river, Nepal—by M. S. Jain.

(vii) Preliminary investigation report of Kothar Dam.

In addition to the above, two other reports were also received and are still under circulation :—

(viii) Report on the Noonthore Dam Site on Bagmati river—by P. K. Chatterjee, Geologist, Government of India.

(ix) Report on geological aspects of the reconnaissance expedition in the Teesta Valley, Sikkim State—by G. N. Datta, Geologist, Geological Survey of India.

As regards Teesta Dam, the Superintending Engineer, Ganga Basin Circle, New Delhi, has informed that the Teesta Dam is a high dam proposed to be of concrete and not an earth dam. Regarding Ramaganga Dam on Ramaganga river, the Chief Engineer, Irrigation Department, Uttar Pradesh has reported that the proposal to form a committee of consultants to advise on the construction of the dam is under consideration and the information could be supplied only after the design of the dam is finalised. The Chief Engineer, Irrigation Works, Punjab has replied that it is not possible to supply the particulars of the scheme titled Pong Dam on river Beas as the scheme is still under investigation and the relevant data is under collection.

A copy of the test results of soil samples of the borrow areas at the proposed dam site at Noonthore in Nepal territory as received from Shri U. N. Jha, Superintending Engineer, was forwarded to the members for information.

As desired by the Committee, the Chief Engineer and Secretary to the Government of Assam, intimated that Shri Sayad Inamul Haque, Executive Engineer was recommended under the Point IV Programme by the State Government for Government of India's considerations.

As decided in the second meeting of the Committee the copies of decisions taken in the first and second meetings were sent to

Dr. Jai Krishna. While acknowledging the minutes of the meetings, Dr. Jai Krishna intimated that he had met Dr. K. L. Rao, Member (D & R) and Chairman of the Committee at Delhi in the last week of February 1960, and was taking steps to prepare the laboratory for experimentations.

The Committee desired at its second meeting that model experiments simulating earthquake effects on structures and dams (especially earth dams) should be undertaken at the Poona Research Station as the experiments would be helpful in arriving at the proper criteria for the design of the dams. Dr. K. L. Rao, Member (D & R) and Chairman of the Committee requested the Director, C.W. & P.R.S. to intimate him the further equipment required for conducting these tests. The Director, C.W. & P.R.S. has sent an estimate amounting to Rs. 3,05,000 for equipment required for the purpose, to the Chairman of the Committee (Dr. K. L. Rao).

It was also pointed out in the meeting that the Council of Scientific and Industrial Research are setting up one of their Regional Research Laboratories at Jorhat in Assam. The Committee considered that this laboratory is very well suited to take up extensive experiments on the subject. It was decided that Council of Scientific & Industrial Research should be approached to equip their laboratory at Jorhat with suitable apparatus to conduct all experiments on earthquake effects on structures and dams. Accordingly Dr. K. L. Rao, Member (D & R) and Chairman of the Committee requested the Secretary to the Council of Scientific & Industrial Research, New Delhi, to equip this laboratory for this purpose with latest and up-to-date apparatus for conducting all expermients on earthquake effects on structures and dams. The Under Secretary, Council of Scientific & Industrial Research has intimated that in the Regional Research Laboratory established by Council of Scientific & Industrial Research at Jorhat, there will be a separate block and a unit for earthquake engineering research provided with the necessary equipment and facilities and the same has been confirmed by the Director, Regional Research Laboratory, Jorhat, Assam.

(c) **Action taken on the decisions of the Third Meeting**

1. A list of the projects under construction and investigation has been collected from Central Water & Power Commission and sent to Dr. Tandon in connection with the preparation of a note on earthquake occurrence, features, etc.

2. Sarvashri Y. K. Murthy, Superintending Engineer, Central Water & Power Commission and I. P. Kapila, Deputy Director, Bhakra Dam Designs, have been requested to prepare a note on (i) the procedure of design of high dams in seismic regions in India, and (ii) the procedure of design of high dams in seismic regions abroad

3. Dr. Jai Krishna has been requested for collection of the data with regard to (*i*) Laboratory Equipments and (*ii*) Field Experiments.

(d) **Action taken on the decisions of the Fourth Meeting**

1. The Chairman, at the outset mentioned that as a result of correspondence some papers were received from Mr. Karbo Samenza

from Italy. These papers have been translated into English and circulated to the members. Some information was also received from Mr. Visentini and Mr. Fronsini from Rome, copies of which have been circulated to the members of the Committee.

2. In regard to the note on the earthquake occurrence, seismic features etc. prepared by Dr. A. N. Tandon, the Chairman explained that the purpose of the note is to consider broadly the maximum seismic intensity which had been recorded during the past 200 years on the various sites on which important river valley projects are proposed to be constructed. He further stated that it is assumed that the intensity in future earthquakes would not exceed the maximum recorded in the past. For this purpose, the various sites have been divided into five regions. Dr. Tandon added that he considered the ground factor to be the most important which has to be determined in the case of each dam.

3. The Chairman decided that the note on the design of earthquake resistant dams in India and abroad to be prepared by S/s Y. K. Murthy and I. P. Kapila be expedited.

4. The Chairman desired that Shri B. R. Chopra, Deputy Director, Central Water & Power Commission should prepare a bibliography on the subject dividing it into two sections; (*a*) earthquakes and general characteristics and (*b*) engineering aspects.

5. It was pointed out that in pursuance of the decisions of the second meeting, Dr. Jai Krishna has prepared a note on the experimental field work for carrying out studies on earth dams in seismic zones and the note has been circulated to all the members.

6. It was noted by the Committee that in regard to sending of an officer of the Assam Government for training abroad, the information received from the Secretary, P.W.D., Assam State was that there was no engineer officer with requisite experience in the design of dams in that department who could be recommended for training in the earthquake engineering abroad.

7. It was also noted that a copy of the publication of Japan Society of the Civil Engineers has since been received in the library of Central Water & Power Commission.

8. With reference to the requirements of funds to the extent of Rs. 1,50,000 for the studies proposed to be carried out by Dr. Jai Krishna at the University of Roorkee, it was brought out for the information of the Committee that on approaching for funds, the Secretary, Central Board of Irrigation & Power has stated that the proposed studies may be done in collaboration with the Board with the funds obtained from the Council of Scientific & Industrial Research; as the Central Board of Irrigation & Power had already earmarked the entire Third Plan funds to the other problems under study in the various research stations.

9. Shri Mookerjea (West Bengal) suggested that it would be useful if a reference is made to the Government of Burma and to the Consulting Engineers M/s Binnie Deacon & Gooley to know if the Gyobju Dam in Burma on which he had worked has since experi-

enced any earthquake effects and if so, its behaviour as a result of the same. It was decided to obtain this information.

10. The Chairman desired that the following points should be incorporated in the final report of the Committee:

(a) A review of the earthquake characteristics relevant to engineering structures.

(b) characteristics and description of Indian earthquakes.

(c) a review of the practices laid down in the foreign countries for seismic design of dams.

(d) recommendation for the structures in India:

(i) Field data

(ii) Experiments

(iii) Design criteria and procedure.

(e) recommendations for specific projects contemplated to be undertaken in the near future.

It was also decided that drafts for items a, b and d(i) will be prepared by Dr. Tandon assisted by Shri I. P. Kapila. Drafts for items c, d(iii) will be prepared by Shri Y. K. Murthy and draft for item d(ii) will be prepared by Shri Guha.

**(e) Action taken on the decisions of the Fifth Meeting**

1. It was brought out that a special meeting of the members of the Committee who were at Delhi was held in the Central Water & Power Commission on 4-1-1963 to review the position regarding the work to be done by the Committee and to decide what should be done to finalize the recommendations of this committee very early. The Chairman explained broadly the discussions held in the last meeting and stated that the main aim of this Committee should be to determine the extent of saving that is likely to result on the basis of specific recommendations to be made by this Committee in regard to the assumptions in the designs to provide against seismic forces.

2. Dr. Guha described the blasting field tests undertaken in the Beas Dam at Pong during the last two years and explained as to how a factor of .15 for horizontal acceleration has been suggested for the purpose of design. He also felt that with further testing, it may be possible to reduce this factor further which will result in a considerably smaller section of the dam. The Committee thereupon decided that copies of the memorandum describing seismic tests carried out at Pong Dam may be circulated to all the members for their information.

3. It was decided that the field and laboratory investigations on the analogy of the tests carried out at Pong Dam should also be carried out for the following four projects, in the first instance:

(a) Ramganga,

(b) Kopili,

(c) Beas-Sutlej Link; and

(d) Kotha.

It was also decided that the following projects should be taken up on second priority for field and laboratory investigations:

(*a*) Barapani, Stages II & III.

(*b*) Teesta,

(*c*) Barak,

(*d*) Gandak; and

(*e*) Jaldhaka.

4. It was also decided that members of the Committee should visit these projects in order to acquaint themselves with the topography, geology etc. and also issue necessary instructions to the field engineers to carry out the necessary drilling for carrying out the fields tests. In this connection, it was also decided that a letter may be issued by the Chairman of the Committee to the various Chief Engineers requesting them to earmark the necessary funds for carrying out the laboratory and field tests and also for providing necessary facilities to the Committee for carrying out these tests.

5. The members of the Committee were requested to take up field studies on the Kopili Dam in the first instance. In this connection, Shri Y. K. Murthy was requested to supply the information regarding the geology of the site and the values of acceleration assumed till now for structures in the nearby areas to the members of the Committee.

6. The Chairman desired to know whether subject to other considerations like availability of suitable material etc. it would be possible for the Committee to advise whether it will be preferable to put in a concrete, masonry or earth dam at a particular site from seismic considerations. Dr. Jai Krishna felt that although the regional geology would decide to a great extent, it should be possible to give some opinion based on experiments carried out on the vibrating table. For this purpose, he pleaded for some funds. The Chairman stated that these will have to be borne by the project authorities from the funds assigned for investigations of projects. It was decided that the work on property of earth materials on vibration may be carried out by Dr. Jai Krishna and Dr. Pais Cuddou of Central Water & Power Commission, jointly.

7. It was decided that an appendix should be prepared giving in detail the lists of equipment available in the various laboratories at Roorkee, Jorhat and Poona in order to assess the programme and quantum of work that could be carried out and to determine whether any further equipment was to be procured. It was decided that Dr. Jai Krishna and Dr. Guha will furnish the lists.

8. It was decided that the recommendations of the Committee should mainly concern concrete, masonry and earth dams and barrages. Other river valley and appurtenant structures may be taken only if time permits.

9. The recommendations should give a guiding criteria for suitable coefficient for design against seismic forces from scientific and technical considerations. In this connection, Shri B. R. Chopra

pointed out that in view of the fact that (*i*) earthquake is a transient phenomenon (*ii*) its nature and effects are far from having been adequately analysed and understood. (*iii*) the dams have generally stood well past earthquakes and causes of damage have been few, the damage being mostly short of failure, a lot of economic consideration is given in selection of the seismic coefficient in the design of dam. In fact, all over the world, factors which are considered in adoption of seismic coefficient for dams include its importance and effects on the downstream reaches. In this connection, he pointed out the following extract from the book "Problems in Earthquake Resistant Design for Civil Engineering Structures published by the Japan Society of Civil Engineers"—"because of the fact that the effects of earthquake specially of earth structures and earth foundations are not precisely known as yet and the designing is inevitably influenced by economy, it may be stated that philosophy of resignation very likely underlies general policy for earthquake resistant design in Japan."

10. It was also decided that only those projects which have been included or are likely to be included in the Third or Fourth Five Year Plans and for which allotment of funds for construction or investigations are available should only be considered by this Committee.

11. It was brought out for consideration of the Committee that in view of the fact that the First Committee set up by the Brahmaputra River Commission in 1956 had taken the view that its deliberations should be confined to the feasibility or otherwise for building dams purely from considerations of the effects of earthquake which include rapid siltation of reservoirs, whether siltation of reservoirs due to earthquakes should also be included as a point for study by the present committee. In this connection, it was also pointed out that Dr. Guha had made a study of this problem for the Kopili Project. It was, however, felt that data and knowledge of this aspect at present are meagre and an effective contribution may not be possible. However, a review of this decision may be done later on if called for.

*APPENDIX A*

## SUMMARY AND RECOMMENDATIONS OF THE TECHNICAL COMMITTEE ON ADVISABILITY OF CONSTRUCTING HIGH DAMS IN SEISMIC ZONES

The Brahmaputra River Commission, at its fourth meeting held at New Delhi on 7th March 1956, constituted a committee to examine the advisability of constructing high dams in seismic zones. In pursuance to this resolution of the Commission, the Committee considered its own terms and reference and decided that, it would confine itself to the consideration of the feasibility of building high dams in seismic areas of Assam and West Bengal purely from earthquake considerations. The factors, which cause rapid siltation of reservoirs would, however, be taken into consideration but other factors or the difficulties of construction etc. would be outside the purview of the Committee.

Seismically, it may be stated that the worst areas are the Shillong plateau and the north eastern corner of the State where the Himalayan range and the Arakan Yomas meet. The seismic conditions, however, improve progressively from east to west along the Himalayas in this region.

The Committee has given consideration to individual dam sites in Assam and West Bengal in the light of the data and other informations available.

In the Kopili Basin, the Kopili dam site is considered feasible, unless some unfavourable features are revealed during the detailed investigations to render the site unsuitable. The result of the detailed investigations conducted so far seems to reveal no favourable feature at this site. The Diyung dam site in the basin has been found to be geologically unsatisfactory. The Barapani and the Killing dam sites in this basin are considered feasible provided, geologically, satisfactory foundations are available.

No dam site has been found on the Barak which can be considered suitable geologically, but seismologically the construction of a dam in this region is considered feasible.

Construction of dams in the Dihing Basin at the tentative dam site on the Noa-Dihing, the Tirap, the Namphuk and the Namchik is considered feasible, provided suitable rock foundations are available at those sites.

If foundation rocks are satisfactory on detailed explorations, the Subansiri dam site would be feasible for constructing a high dam of the height contemplated, provided due allowance is made for the high ntensty oif earthquake shock experienced in this region. It may, however, be pointed out that the Subansiri catchment area is liable to heavy landslide during earthquakes which may lead to rapid siltation of the reservoir.

The Jia-Bhareli dam site, for the height of the dam contemplated, has not been found suitable geologically, apart from its being in highly seismic area.

The Dihang, the Dibang and the Luhit dam sites lie in an area which was the worst affected during the 1950 earthquakes. Here, the intensity of shock had reached X of M. M. scale. There were very heavy and extensive landslides in this region during the 1950 earthquake, and the area is yielding an abnormal silt load in the rivers. Dams on these three tributaries of the Brahmaputra should be built only if these be considered imperative, and if geological conditions at all permit.

The Manas dam site is situated in a area which seismologically is as active as any other along the Himalayan boundary fault region. The site has not yet been investigated geologically and the area has not even been geologically mapped. Before any large scale investigations are undertaken at this site, a careful geological reconnaissance should be carried out first and if suitable rock foundation is available at the dam site, the construction of a dam would be feasible seismically.

The Kulsi dam site lies in an area which, seismically ,is the worst affected in this region. In the great Assam earthquake of 1897 the destructive intensity of XI on M. M. scale was reached here and small objects were thrown up in the air. This site which has been found otherwise uneconomic, is considered not feasible on seismic considerations.

The area in West Bengal in which the Tista, the Rangit, the Lish, the Gish, the Chel and the Jaldhaka on which dams have been proposed is seismologically not as bad as the area in the eastern part of Assam, provided suitable rock foundation is available at these dam sites. Construction of dam of the height contemplated is considered feasible.

In Assam and in upper parts of Burma severe earthquakes are fairly frequent and violent. Apart from the consideration of safety to dam structures, the life of reservoir has an important bearing in taking decisions regarding the advisability of construction of dams in these regions. Landslides are common and extensive, particularly in the outer Himalayas, where soft rocks of younger formations predominate. and as a result colossal quantities of detritous and silt flow into the rivers, contributed mostly by these landslides and therefore the life of the reservoirs created in such hilly tracts would be comparatively shorter.

*APPENDIX B*

# SEISMIC STABILITY OF EARTH DAMS*

by

E. E. ESMIOL

## STATEMENT OF PROBLEM

Though most of the problems inherent in the design of earth dams have been wholly or partially solved, the problem of applying an "earthquake factor" to an earth dam design has been given little specific consideration, and earthquake design criteria have not been delineated. As a supplement to present knowledge pertaining to earth dams, a general study of earthquake characteristics and their effect on earth materials and structures is presented in this memorandum to aid the development of methods of analysis and design criteria.

## INTRODUCTION

With the rapid advances in the science of soil mechanics and the resultant improvements in the methods of soil investigation, moisture control and compaction, stable earth dams are being built on sites that have been considered unfavourable for the construction of concrete dams. The economic and technical advantages enjoyed by earth material construction is increasing the number of earth dams being built. Many of these structures will be situated in active seismic areas and the factor of seismic stability must be given a measure of consideration.

Earthquakes, vibrations, or oscillations of the surface of the earth have presented almost insurmountable problems with respect to all types of structural design. Only recently have modern engineering and testing techniques fostered the rational design of earthquake-resistant structures. The application of an earthquake design factor to earth dams involves special problem posed by the nature of earthquakes, by the characteristics of earth materials, and by the geology of the area in which the structure is located. Design features (density of the compacted embankment, reservoir water surface, length of the structure, height of the structure and side slopes of the embankment) must be considered in relation to the resonance and damping of the structure and to the relative intensity and acceleration of the earthquake, and to dominant ground frequencies of the area. Except in rare instances, design requirements will be controlled by the purpose of the structure, dam site topography, foundation conditions, and reservoir operation, as well as the earth materials available. In general, these requirements have been given prior consideration over designs involving seismic stability.

## EARTHQUAKES

### Definition

An earthquake is an oscillation or vibration of the surface of the earth caused by a transient disturbance of the gravitational or elastic equilibrium of the rocks at or beneath the surface.

*Extracts from Technical Memorandum 641 of USBR.

The source of the earthquake is called the focus of the shock, or focus. The area on the surface directly above the focus is called the epi-centre.

### Tectonic Earthquakes

Earthquakes that result from a sudden deformation of the earth's crust by either faulting or warping and are clearly structural in nature, are called tectonic. This type of earthquake is most prevalent and causes the greatest and most widespread damage. To this class belong all earthquakes during which visible displacements are noted along a high-angle fault trace or scarp. These displacements may be horizontal, vertical or lateral with reference to the plane of the fault. Earthquakes caused by the relief from elasticstrain are prominent throughout Oregon, Washington, California, Nevada and Alaska. Still of tectonic nature are the displacements along low-angle thrust faults, interbed faults, and differential movement within the earth. These displacements are not generally measurable, but cause earthquakes of sufficient magnitude to damage structures.

### Wave Types

Three predominant types of waves are recognized and recorded, longitudinal, transverse and ground. Ground waves are further subdivided into Rayleigh and Love waves. Longitudinal waves are the compressional waves in which the particle vibrates in a plane perpendicular to the wave front and in the direction wave front is travelling. This wave is usually designated at "P" or primary wave because it has the greatest velocity. $\bar{P}$ designates a primary wave in the sial layer and Pn, a wave in the sima layer. Transverse waves are shear waves in which the particle vibrates in a plane perpendicular to the wave front and in a direction perpendicular to the $\bar{P}$ wave. The shear wave is designated as "$\bar{S}$" or secondary wave, because its velocity is slower than the $\bar{P}$ wave. The ground or surface waves, also called long waves, are slower than either or $\bar{S}$ waves. Rayleigh (R) waves are ground waves that travel along the surface of the earth and the oscillation of the vibrating particle forms an ellipse with the major axis of the ellipse in the direction of the wave. Love waves have no vertical component and vibrate in a plane perpendicular to the Rayleigh waves.

Many combinations of these waves are recognized, but engineering problems are concerned primarily with P. S. and R waves. Average values for the velocities and periods of these waves are shown in Table 2.

TABLE 2

*Average Wave Velocities*

| Wave | Velocity (miles per second) | Period (seconds) |
|---|---|---|
| $\bar{P}$ | 4·5 | 1/2 to 6 |
| $\bar{S}$ | 2·7 | 11 to 13 |
| R | 2·0 to 2·5 | to 40 seconds, maximum 60 |

**Elastic Constants**

Since these waves are generated by the release of strain energy and travel through elastic media obeying Hook's law, they are related by these formulae :

$$\tau = \frac{E\,\sigma}{(1+\sigma)\ (1-2\sigma)} \qquad (1)$$

$$\mu = \frac{E}{2\ (1+\sigma)} \qquad (2)$$

$$V\bar{P} = \sqrt{\frac{E\ (1-\sigma)}{\rho\ (1+\sigma)\ (1-2\sigma)}} \qquad (3)$$

$$\frac{V\bar{P}}{V\bar{S}} = \sqrt{\frac{2\ (1-\sigma)}{1-2\sigma}} \qquad (4)$$

$$V\bar{S} = \sqrt{\frac{\mu}{\rho}} \qquad (5)$$

$$VR = \cdot 9914\sqrt{\frac{\mu}{\rho}} \qquad (6)$$

$$\beta = \frac{3\ (1-2\sigma)}{E} \qquad (7)$$

$$\sigma = 1/2\ \frac{\left\{\dfrac{V\bar{\rho}}{V\bar{S}}\right\}^2}{\left\{\dfrac{V\bar{\rho}}{V\bar{S}}\right\}^2} \qquad (8)$$

$$\mu = (v\bar{S})^2\rho \qquad (9)$$

where

$\tau$ = Lame's constant,

$\mu$ = Modulus of rigidity,

$\sigma$ = Poisson's ratio,

E = Young's modulus (modulus of elasticity),

$\rho$ = density,

$\beta$ = cubic compressibility (bulk modulus),

$V\bar{P}$ = velocity of primary wave,

$V\bar{S}$ = velocity of shear wave, and

VR = velocity of Rayleigh wave.

**Earthquake Intensity**

A completely satisfactory method of evaluating the intensity of earthquakes with respect to engineering structures has not been determined. Several scales have been, proposed, *viz.*, Rossi Forel scale, Modified Mercalli scale 1931 and the Mercalli-Cancani Seiberg scale.

The following table 14 gives the values of the Mercalli-Cancani-Seiberg scale :

TABLE 14

*Mercalli—Cancani—Sieberg*

| Description | Accel. mm/Sec². | Grade |
|---|---|---|
| Not felt | 0-2·5 | I |
| Very feeble | 2·6-5 | II |
| Feeble | 6-10 | III |
| Moderate | 11-25 | IV |
| Fairly strong | 26-50 | V |
| Strong | 51-100 | VI |
| Very strong | 101-250 | VII |
| Ruinous | 251-500 | VIII |
| Destructive | 501-1000 | IX |
| Very destruction | 1001-2500 | X |
| Great destruction | 2501-5000 | XI |
| Total destruction | 5001-10000 | XII |

While Fig. A-1 gives the correlation between the other two scales.

To the engineer, one of the most practical methods of determining earthquake intensity is the isoseismal map which shows the areas of equal intensity adjacent to the epicentre.

## EFFECT OF EARTHQUAKE WAVES ON SOILS

**General**

The ground on which a structure is built has an important bearing on its stability. Destruction is less on hard rock than on soft, and is worse on unconsolidated ground, especially if the soil mantle is thin (under 100 feet). As elastic waves enter the less consolidated material at the surface, the amplitude becomes greater while the velocity and wave length diminish. During severe earthquakes, the amplitude of surface waves has been estimated at 1 to 2 feet, the wave length at 50 to 100 feet.

Semi-consolidated soil of any type, particularly lacustrine, deltaic and flood plain deposits with a high proportion of clay and silt, will settle more than deposits of gravelly material. Fine silts and clays may be liquefied temporarily and erupt through or flow.

**Some of the Dams designed for Seismic Stability**

(1) Coyote Dam in Santa Clara Valley near Sanjose California.—The dam is 140 ft. high, 900 ft. long and stores 40,000 ac. ft of water. The dam is bisected by the trace of the Hayward fault.

(2) San Andreas Dam—This dam along the San Andreas fault withstood 1906 earthquake. It is a rolled earth dam with puddle clay core. It is 95 ft. above river bed and 130 ft. above cut-off trench and 970 ft. long. The cut-off trench is 20 ft. wide and extends to a minimum of 45 ft. into the rock foundation.

(3) Gyobyu Dam.—This is across Gyobyu Chaung, 55 miles north of Rangoon, Burma, max. height 134 ft. length 700 ft. and volume of fill 3,70,000 cu. yds. It has an R. C. core wall 7' -6" thick at bottom and 4'-0" at top.

## FACTORS AFFECTING THE STABILITY OF EARTH DAMS

In applying an "Earthquake Design Constant" to the earth dam, the problem is complicated by the nature of earthquakes, the elastic constants of the soils comprising the foundation and embankment, the height of the structure, geographical location of the structure, reservoir water surface elevation, and pore water pressure. Any adequate single solution of the problem would be the result of a rather complicated juggling process. When analyzing the seismic stability of the earth dam major consideration must be given to the effect of elastic waves on (*a*) possible increase of pore water pressures in the embankment, (*b*) resonant frequency of the structure and its foundation, (*c*), increased stress applied to the embankment from the reservoir load, and (*d*) increased stress in the embankment proper. In addition, solution to these problems should be capable of being applied to the standard slip circle method of analyzing the stability of earth dams. However, the problem of resonance, by its nature, cannot be applied to the slip circle method analysis.

## A PROCEDURE FOR INVESTIGATING SEISMIC STABILITY

Initially, stability studies should be divided into two groups that are based entirely upon the location of the dam, the local geology and local seismicity. These classes would be (*a*) those dams whose axes will intersect the trace of a known fault where measurable displacements have occurred and will probably reoccur in the future and (*b*) those dams that will be built in seismically active areas where they will be subjected to earthquake shocks but where the possibility of vertical or horizontal displacement of one section of the dam with respect to the other is remote.

The number of dam to be built under Group (*a*) wil be limited. Should it be necessary to build a dam under such conditions, a thorough study should include (*a*) fault displacements both horizontal and vertical, (*b*) the expected intervals between earthquakes of maximum intensity, (*c*) the predominant frequency of the elastic waves, (*d*) the estimated total cumulative displacement for a selected period of time and (*e*) geological conditions at the dam site. The design criteria mentioned previously would be applicable. In this case, stability would not be governed entirely by the physical properties of the construction materials, because it is assumed that they would fail in shear. The physical dimensions of the structure must be chosen so that under the most severe conditions, the water stored in the reservoir will not find a route of escape through,over, or under the dam.

More earth dams will be constructed under the conditions of Group (*b*) Geographically, these dams will be limited to California, Oregon, Washington, north central Montana, southern Utah, southwestern Nevada and western Colorado. Further, those areas experiencing earthquakes with intensities of less than Degree VII, modified Mercalli scale, need not be considered.

## METHODS FOR DETERMINING SEISMIC STABILITY

### Application to Modified Method of Slip Circle Analysis

The dynamic action of a dam may be converted to a problem of statics if inertia forces, expressed as mass multiplied by the acceleration reversed, are introduced. Thus, on each part of the structure weighing 1 pound, there is a horizontal inertia force diametrically opposite to the direction of acceleration and expressed:

$$Fx = \frac{1\ \text{lb}}{g} \alpha g = \alpha\ \text{lb} \tag{17}$$

where

Fx=force in lb.

g=acceleration of gravity, 32.2 ft./sec.$^2$, and

$\alpha$ =ratio of intensity of the earthquake and equals maximum harizontal component of acceleration of the foundation divided by g.

The measure of earthquake intensity most commonly used is the force in pounds generated by the acceleration and is calculated by dividing the horizontal or vertical component of acceleration in the elastic material by the acceleration due to gravity. This earthquake factor is a ratio of accelerations. The earthquake factor for an earthquake of graded intensity may be found by dividing the accelerations listed in the Mercalli-Cancani-Sieberg scale of earthquake intensities (Table) by 10,000 mm/sec.$^2$. Vertical acceleration, when applied, is normally considered to be one-third the horizontal acceleration. Also, percentage of gravity may be determined from Cornwell's curve, (Figure A-1) where intensity is given by one of the sensory intensity scales.

Vertical and transverse components of force can be ignored if the earthquake intensity is less than Grade IX, modified Mercalli scale, because these components will be very small as compared to the horizontal components. If earthquake intensities of IX or greater are expected the source of the waves will be very close to the structure and all waves will arrive almost simultaneously, then the vertical component will be roughly one-third of the horizontal component.

H. M. Westergaard derived formulas expressing reservoir water thrust on the dam. These formulas were developed for a dam with a vertical upstream face but may be applied to a sloping face by modifying the values as a function of the slope angle. Maximum pressure of the water on the dam due to the horizontal component

$$P = (2B-1)\ \frac{T}{2\pi}\ gEw \tag{18}$$

where

B, a constant=½ (where waves moving towards the dam predominate over waves moving away from it).

P=maximum water pressure on the dam at any depth.

T=period of horizontal vibration of the foundation.

E=modulus of elasticity of a volume of water 300,000 lb./in.$^2$ or 21,600 ton/ft.$^2$, and

w=weight of water per unit volume 0.3125 ton/ft.$^2$

C. N. Zangar has modified Westergaard's original formulae by electric analogy methods and developed curves. Figure A5, for pressure coefficients for uniformly sloping faces. This curve shows that the earthquake force decreases with an increase in slope angle. Through the positive one-half cycle, the thrust due to reservoir would be a compressive stress; through the negative half cycle, a tensile stress.

Experiments by Casagrande indicate that cohesive material may be twice as resistant to transient dynamic stresses as to static stresses. Since the "shear angle" (assumed to be equivalent to the angle of internal friction) is nearly constant, the cohesion must increase. Thus, it would seem valid to assume that double cohesion could be applied to the equation for safety factor in the modified slip circle method of stability analysis.

The slip-circle method of stability analysis is based on W. W. Daehn's modification of the Swedish method of analyzing earth slopes. The method is based on the assumption that the failure plane of an earth slope is cylindrical. The resulting factor of safety is a ratio of total shearing resistance to total shearing force.

The force exerted by any segment within the circle is equal to the weight of the segment and acts vertically downward through its centre of gravity. Normal force and tangential (shearing) force are determined by completing the force triangle with lines in the normal and tangential directions. The safety factor is expressed.

$$SF = \frac{C + \tan\theta\,(\Sigma N - P)}{\Sigma T} \tag{19}$$

where

SF=safety factor,

C=cohesion,

$\tan\theta$=tangent of the angle of internal friction,

P=pore pressure,

$\Sigma N$=sumption of normal force, and

$\Sigma T$=sumption of tangential force.

For earthquakes where the acceleration is not greater than 0.1 g, the first impulse to strike the structure would be the P wave, unless the epicentre was very near to the structure, than P, S and P waves would arrive simultaneously. A shock of this intensity, acting for a short period, would not transfer enough energy to the earth structure to effect a change in volume sufficiently large enough to cause a charge in pore pressure. Most of the energy in the waves would be absorbed by damping and inertia of the earth material. in an earthquake of high intensity, the R waves would predominate over the $\bar{P}$ and $\bar{S}$ waves and a volume change would be effected by compression of the soil and by rearrangement of the soil particles. When the embankment is compressed, the pore water pressures will increase. Because the increase in stress is applied as a static load, the pore pressure may be estimated as a per cent of the weight of material above the slip circle by J. W. Hilf's modification of J. H. A. Brahtz's equation where $\Delta$ is taken as the amount of consolidation due to increased stress. The equation would be written.

$$P+dP=\frac{Pa\ (\Delta+d\Delta)}{Va+hVw-(\Delta+d\Delta)} \qquad (20)$$

where

P=pore pressure,

Pa=atmospheric pressure,

Va=initial free air content,

$\Delta$=consolidation,

Vw=specific volume of water, and

h=0.0197

In applying the earthquake factor to a zoned earth dam (Figure A6) a horizontal acceleration of 0.1 g is assumed as a representative value for earthquakes classified in intensity Grades VIII and IX on the modified Mercalli scale of 1931. The stress increase of 0.1 of the weight of material above the slip circle, acting horizontally. The horizontal earthquake force is resolved to the slip circle and the normal and tangential components added algebrically to those from the weight of material. Since all values are in cubic feet of water or earth, the force area is easily outlined by scaling the change in stress on the verticals. Force areas are planimetered and total forces determined by multiplying the planimeter reading by the planimeter constant and the unit weight of the materials.

For the section of the zoned earth dam considered, Figure A6, these values are assumed:

Wet density

Zone 1=127.9 lb./cu. ft.

Zone 2=137.5 lb./cu. ft.

Cohesion

Zone 1=800 lb./sq. ft.

Zone 2=none.

Tan $\theta$

Zone 1=0.65

Zone 2=0.70

Zone 1 material is impervious silty clay and zone 2 material is pervious sandy silt. Riprap is considered as Zone 2 material.

On the slip circle, the radius is 165 feet and the effective arc through which cohesion acts is 100° (288 feet).

**Pore pressures were calculated from the Hilf-Brahtz equation and Figures A7 and A8. The effective reservoir thrust was calculated from Zangar's curve Figure A5.**

Three causes are considered : In Case I, stability is calculated for the completed structure where the materials were placed 2 per cent dry of optimum. There is no reservoir water thrust and no earthquake. For Case I :

$C=230$ Kips,
$\Sigma N=1{,}712$ Kips,
$\Sigma T=452$ Kips,
$P=140$ Kips, and
$SF=2.77$.

Case II is the same as Case I, except a factor of 0.1 g horizontal loading due to earthquake is superimposed on the static loading above the slip circle. The earthquake is assumed to be acting on the element of the section above the slip circle. Here, the normal forces decrease the tangential forces increase, and the pore pressure is constant. For Case II :

$C=230$ Kips,
$\Sigma N=1{,}492$ Kips,
$\Sigma T=507$ Kips,
$P=140$ Kips, and
$SF=2.17$.

In Case III, the effect of reservoir water thrust is considered relative to the conditions stated in Case II with these results :

$C=230$ Kips,
$\Sigma N=1{,}746$ Kips,
$\Sigma T=320$ Kips,
$P=140$ Kips, and
$SF=3.98$.

Sample calculations and illustrations for the three cases are shown in Figure A6. For the cases cited, pore pressure and cohesion were assumed to remain constant. Recent limited studies indicate that cohesion may double when transient stresses are applied to cohesive earth materials. If the cohesion is doubled, the safety factor becomes 2.59 for Case I with soil compacted 2 per cent dry of optimum moisture. In the light of field observations of railroad and highway fills and earth dams subjected to earthquake, probably cohesion does increase particularly in moist soils.

Since the increase in stress is applied as acceleration reversed, an earthquake coming from the upstream direction would be the worst condition affecting the dam. Pore pressure would increase or remain constant, tangential forces would increase and normal forces decrease relative to a slip circle on the upstream face.

**Dynamic Method**

Maximum destruction results when the natural frequency of the structure equal the prevailing frequency of the earthquake elastic waves. In addition, to produce the effect of resonance, the structure must have little damping. To evaluate resonance, definite information concerning (*a*) damping properties of the structure, (*b*) elastic properties of the structure, and (*c*) relative ground frequencies are necessary. Both Hailand and Mononobe have succeeded in vibrating models of earth dams. These experiments indicate that the earth dam would be subject to a force two or three times stronger at the crest than at the base.

Magnification (W) at resonance or increase in amplitude is given by the equation:

$$W\ max = \frac{1}{2n}\sqrt{1-n^2} \quad \text{or (approx.)} \quad \frac{1}{2n} \tag{21}$$

n=relative damping

and indicates that damping above all other conditions is most important in considering the seismic stability of an earth dam.

In calculating the frequency of an earth, it is considered as a cantilever whose base is greater than its height. Then, the period varies directly with height, inversely as the square root of the density, and directly as the velocity of the $\bar{S}$ waves, so:

$$fo = \frac{2\cdot 4048}{H}\sqrt{\frac{\mu}{P}} \tag{22}$$

fo=frequency of the dam, and
H=height.

A comparison of the structure with the frequency of the earthquake waves will give a ratio of seismic stability expressed by Hailand as "the ratio of ultimate strength to stress". Note that Vs, so frequency is a function of $\bar{S}$ wave velocity and height. The equation of seismic stability is expressed:

$$Rs = \frac{U}{S} = \frac{UZo}{\mu A}\sqrt{\frac{(fo^2-f^2)^2+4n^2f^2}{f^2}} \tag{23}$$

where

Rs=seismic resistance,
U=ultimate shearing strength,
S=shearing stress=$\mu\theta$,
Zo=elevation of centre of oscillation,
A=maximum ground amplitude,
$\mu$=modulus of rigidity,
f=frequency of ground,
fo=frequency of dam,
n=damping factor, and
$\theta$=tan angle of internal friction.

In addition, amplitude, natural ground frequencies, damping, and modulus of rigidity can be determined by seismic methods or from earthquake records.

If a dam is resonant to natural ground frequencies, methods may be used to change the frequency of the dam; the density of the embankment may be increased, the crest elevation may be raised or lowered, or the damping modified. If these factors cannot be changed the structure will probably fail only near the crest through longitudinal and transverse cracks.

### Earthquake Analyzer

The Bureau of Reclamation has developed an earthquake analyzer in which the resonant characteristics of a structure can be compared with the record of a known earthquake. This machine is constructed around a torsion pendulum so the rotatory displacement represents the translational displacement of a structure. The analyzer automatically registers the differences between the two displacements and draws the relative displacement curve. If several modes of vibration are possible the response for each mode may be determined.

## SUMMARY

To present the complex nature of the problem of developing an "earthquake factor" to be used in the design of earth dams, seismic stability has been examined in this memorandum with emphasis on the types and fundamental characteristics of earthquakes. The data presented represented both theoretical considerations and field observations of the mechanics of earthquakes, their origin, their intensity and their wave trains. Since some dams have been designed to resist elastic wave forces, a description of those structures that have withstood earthquakes has been included with a tentative statement of design criteria. Application of an "earthquake factor" must include a knowledge of the dynamic properties of soil and the observed effects of elastic waves on soils. (Though incomplete, data pertaining to these properties were included to form a basis for possible future investigations). After examining the factors involved in a seismic stability analysis, a procedure for investigating the stability of earth dams was suggested and the adaptation to the slip circle method of analysis was given. The condition of resonance was considered separately because resonance is the phenomenon which, of itself causes the most widespread damage.

## CONCLUSIONS

Earthquakes of intensity VIII or greater (modified Mercalli scale) will undoubtedly cause some damage to earth dams. Though appurtenant structures may be destroyed, damage to the embankment should be limited to settlement of the embankment or foundation and to longitudinal cracks in the embankment near the crest of the dam.

A completely satisfactory method of evaluating seismic stability of earth dams has not been developed.

The components of stability analysis: reservoir thrust, pore pressure; unit density and cohesion, can be resolved to the modified slip circle method of stability analysis for a unit change in stress. An assumption that cohesion will increase 1.5 to 2.0 times the original value for cohesive soils seems justified in the light of existing evidence.

The possibility of resonance is the greatest menace to stability. To a limited degree, the probability of structural resonance with prevalent ground frequencies can be mitigated by changing the height of the dam, by increasing the density of the embankment, or by increasing the relative damping coefficient. Of the three methods, increasing the damping seems the most practical because the pervious outer section of the dam would have a high order of friction damping and increasing the volume of the outer zones and flattening the slopes would, in effect, buttress the core of the dam.

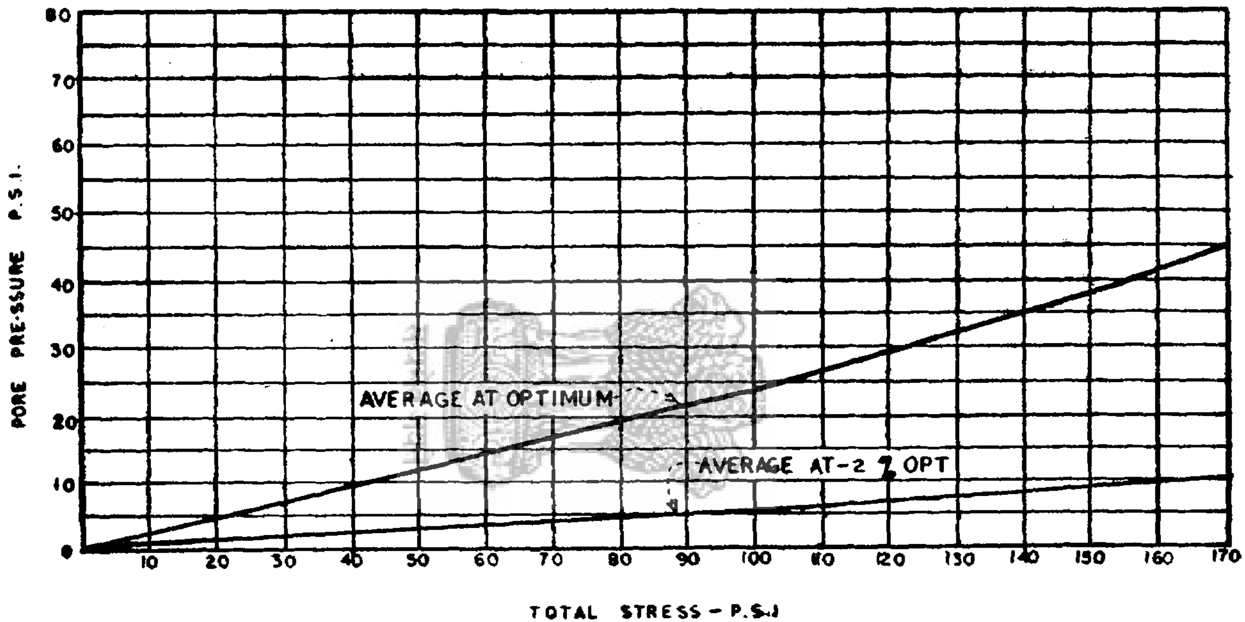
Relative damping is the factor that could control resonance of earth dams. It is suggested that a more through study be made of the friction type of damping in earth materials.

Dynamic properties of soils can be determined in the field by seismic methods.

Earthquake intensity would be the most difficult variable to evaluate, since most data are based on sensory scales.

Prevalent ground frequencies are developed in each area. These prevalent frequencies will vary with the character of the overburden and the rock mantle.

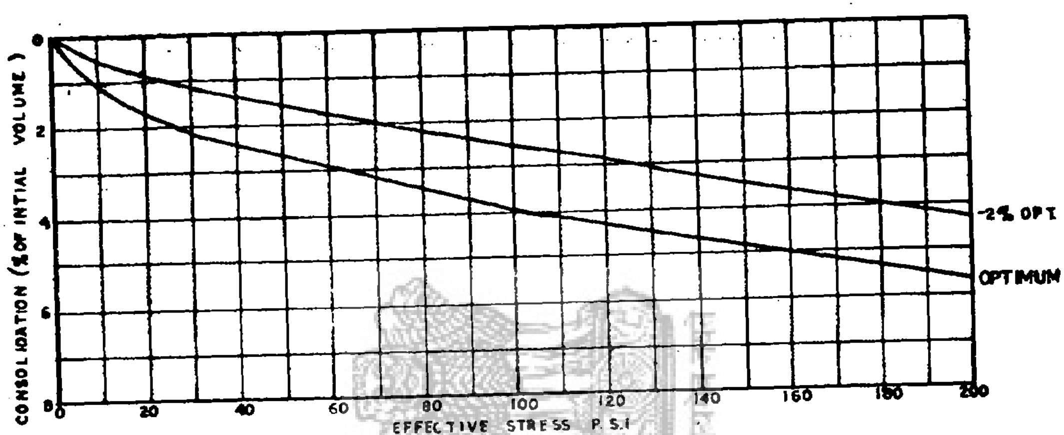
FIGURE A 7




SEISMIC STABILITY STUDY PORE PRESSURE CURVES
PRODUCED FROM TECHNICAL MEMORANDUM 641 OF U S B R

T.BY - B C LENKA

FIGURE A 8




PLACEMENT CONDITIONS

| CURVE | AVE. DRY DENSITY | AVE. % MOISTURE |
|---|---|---|
| -2% OPT | 108.71 | 13.60 |
| OPTIMUM | 110.66 | 15.42 |

SEISMIC STABILITY STUDY CONSOLIDATION CURVES

REPRODUCED FROM TECHNICAL MEMORANDUM 641 OF U.S.B.R

T. BY :- D.C. LENKA

*APPENDIX C*

## SEISMIC CONSIDERATIONS IN DESIGN OF DAMS

by

C. D. MITRA

An earthquake is an irresistible motion of the ground in form of waves which are very often complex in nature and consist of longitudinal and transverse vibrations in earth. The waves generally consist of a first impulsive shock followed by a series of rapid oscillations of a complex pattern. There have been a lot of investigations in the accelerations produced by seismic forces and values of acceleration produced by these shocks have been found to vary to a great extent. This acceleration is usually designated by its ratio to "g" acceleration due to gravity.

According to "Earthquake-Resistant Construction" by Dewell H.D. Published in Engineering News Record in April 1928 "Acceleration ranging from 0.00037 g to 1.000 g have been observed or estimated in earthquakes. An intensity of 0.4 g or more than 12 feet/sec.$^2$ is not uncommon in great shocks". In the Bihar earthquake of January 1934, the following horizontal accelerations were estimated :

| | | |
|---|---|---|
| Monghyr | 11 ft/sec$^2$ i.e., 0.34 g approximately | |
| Patna, Bhagalpur & other places | 3 ft/sec$^2$ i.e., 0.1 g approximately to 6 ft/sec$^2$ | to 0.2 g approximately |

In addition to vibration in the horizontal direction resulting in horizontal accelerations as mentioned above vibrations in vertical directions resulting in vertical accelerations have also been observed during earthquakes, whose magnitudes have been found to be 10% to 20% only of the horizontal acceleration.

Apart from horizontal and vertical vibrations as mentioned above it was pointed out by Mr. Ayre, R. S. that in the most generalised pattern of vibrations in the ground there would be an association of rotatory oscillations of the system with that of usual translatory vibrations. This is produced due to the non-coincidence of the centre of mass with the centre of rigidity of the system.

Damage to a structure during earthquake is mostly caused due to vibrations in earth in horizontal direction. The destructive effect of vertical acceleration on a structure except in dams higher than 150 ft. is small as every structure is designed with a factor of safety in the foundation pressure. The effect of rotatory oscillation also is small on a structure due to its rigidity and size.

When the ground, beneath a structure is suddenly moved to one side, the structure will tend to remain in its original position due to its inertia. The horizontal seismic force "F" applied to the base of

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the structure will be equal to $\frac{w}{g}a$,

where

a=horizontal acceleration in feet per sec. per sec. due to earthquake,

w=weight of the structure in pounds above base or horizontal section under consideration, and

g=acceleration due to gravity in ft. per sec. per sec.

Expressing the horizontal seismic acceleration "a" by its ratio to acceleration due to gravity "g" the horizontal force due to earthquake will be $F=w\alpha$, where $\alpha=a/g$.

In seismic regions dams are likely to be subjected to the following vibratory forces in addition to static forces which are generally considered for non-earthquake areas:—

(a) Due to vibration of dam itself in horizontal direction.

(b) Due to vibration of dam itself in vertical direction.

(c) Due to vibrating column of water immediately on the upstream of the dam.

(d) Due to vibration of tail water, if any.

(e) Due to increase in the horizontal vibration of the dam due to resonance.

(f) Due to movement of silt and ice on upstream of the dam.

(g) Due to movement on faults.

Vibrations due to earthquakes may occur in any direction and maximum destruction to structures is caused when they act horizontally. Separate considerations may be made for (I) Masonry or Concrete Dams, (II) Earth Dams.

## (I) MASONRY OR CONCRETE DAMS

**(a) Vibration in Horizontal Direction**

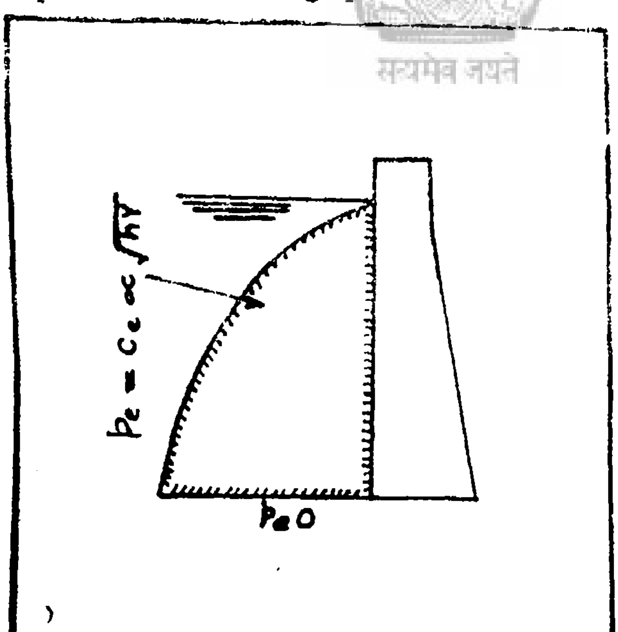
For a gravity dam the most unfavourable direction of vibration will be normal to its axis. When the reservoir is empty the affect on the dam will be worst when the vibratory motion of the dam is from downstream to upstream direction. As usual the dam may be divided into suitable blocks and the earthquakes forie Fe on each block will be equal to $\cdot$W, where W represents the weight of each block. The force will act horizontally through the centre of gravity of each block. The effect of this seismic force will tend to shift the resutant force on the horizontal section under consideration to the upstream side and consequently outside the upstream middle third point. To safeguard against this effect the upstream profile of the dam will need flaring out at the base to keep the resultant force within the middle third. With the reservoir full the effect on the dam will be worst when its vibratory motion is from upstream to downstream as this will tend to shift the resultant force beyond the downstream middle third. Normally, the downstream profile of the dam near the toe is flared out in actual practice beyond the theoretical profile as per conventional practice which automatically keeps the resultant force within the downstream middle point. Although as a safety measure against earthquake the upstream profile of the dam has been proposed to be flared out yet it would be pertinent to mention that such a measure will tend to shift the downstream middle third point more towards the upstream side with disadvantage. This precaution is necessary only when a dam is expected to remain empty after its completion. Once the reservoir is filled with water the upstream flaring out will no longer serve any useful purpose.

**(b) Vibration of the Dam itself in Vertical Direction**

As already stated the intensity of the vertical acceleration will normally range up to 0.25 g, and will have most unfavourable effect on a dam when the structure is vibrating from downwards to upwards. The maximum stresses at the toe and heel of the dam will get multiplied by $(1+\alpha)$, i.e., $(1+0.25)=1.25$. Normally, the increase in stresses due to this effect will be small which will be covered by the usual factor of safety adopted in the design. Similarly, when the structure will be moving from an upward to downward direction the weight of the body will get decreased and stresses in the dam will get reduced, i.e., multiplied by a factor $(1-\alpha)$, i.e., $(1-0.25)=0.75$.

**(c) The Vibrating Column of Water immediately on the Upstream of the Dam**

The inertia of water in the reservoir due to motion of the earth produces a force on the dam, the determination of which is complicated. The nature of water pressure caused by earthquake may be represented by a curve, the true equation of which is complex. Mr. Davis assumed this curve to approximate an ellipse in his book on Applied Hydraulics. Prof. Westergaard solved this issue more correctly and determined that it approached a parabola more than an ellipse as shown in the graph below:




As a matter of fact assuming the curve to be a parabola it becomes simpler to use and the resulting values are on the side of safety. According to Westergaard the pressure at a depth on the dam may be represented by the equation

$$pe = C_e \alpha \sqrt{hy}$$

where $C_e$ is a coefficient, developed by Prof. Westergaard and is equal to

$$Ce = \frac{51}{\sqrt{1 - 0{\cdot}72\left(\frac{h}{1000t_e}\right)^2}}$$

where

h represents height of the dam,

te is earthquake period of vibration, and

$\propto$ is the ratio $\frac{\text{earthquake acceleration}}{\text{acceleration due to gravity}}$

and y is depth below free water surface at which the pressure is determined by Pe.

The total pressure at a depth y, i.e., Pe is given by the expression 2/3 $C_e \propto y\sqrt{hy}$.

The moment due to total earthquake water pressure at any section can be determined by the equation

$$P_c x = \frac{4}{15} C_e . \propto . y^2 \sqrt{ph}.$$

For usual conditions with dam heights not exceeding 250 feet the approximate value of Cê works out to 52. For dams higher than 500 feet the value of Cê has to be determined with caution. The above mentioned equations are based on the assumption that the upstream face of the dam is vertical and normal to the direction of earthquake motion. This condition is usually satisfied by a straight gravity dam. In case of an oblique earthquake motion, where $\theta$ is the angle between the direction of motion and normal to the face of the dam at the point under consideration the above mentioned equations need only be multiplied by Cos$\theta$. In a paper written by Mr. Fergusson, F. F. "Earthquake Effects on Dams" published in the C.B.I. Journal of May 1959 necessary values for the inertial forces due to masonry and water have been worked out for dams varying from 100 feet to 250 feet in height, (assuming to be ranging between 0.05 and 0.175) in a tabular form for ready use by engineers. To make a structure safe against above mentioned inertial effect of water this force is taken into account like other forces acting on the dam.

**(d) Vibration of Tail Water, if any**

The existence of tail water will add to the stability of the dam in an earthquake motion. As such its effect may not be taken into consideration.

**Increase in Vibration owing to Resonance**

Every structure has its own free period of vibration depending upon its height, modulus of elasticity and length of base. When the earthquake oscillating force applied at the base of a structure has a time period almost equal to the free period of vibration of the structure, a dangerous cumulative effect may be produced. Prof. Westergaard has determined this time period for a concrete gravity dam of triangular section considering reservoir empty, as denoted by the equation

$$t_s = \frac{h^2}{2000a}$$

where

ts is the time period of vibration in seconds,
h is the height of the dam in feet, and
a is the length of the base in feet.

The modulus of elasticity has been taken as
$2 \times 10^6$ lbs./sq. inch.

It will appear from above mentioned relation that the possibility of resonance effect on dams need not be of material importance for heights less than 1000 feet for reasonable ratio of h to a.

The pattern of vibration of a clamped free elastic bar with one end fixed in foundation was investigated by Rayleigh and Timoshenko applying this analogy in the case of a dam its time period was determined by Messrs. S. K. Guha and Gurdas Ram by solving a differential equation and was published in a paper "Vibration of Dams—their free period". In a similar method the dynamical analogy of a wedge fixed at the base was also considered with symmetrical sides about the vertical axis by the said authors and its time period of vibration was mathematically determined. A table was prepared showing the comparative values of natural periods of vibrations of a few high dams in India and abroad from which it will appear that the values obtained according to Prof. Westergaard's & Wedge methods compare favourably with each other, but differ widely from those given by the bar analogy method. In conclusion it may be said that damage to dams due to direct resonance are considerably low.

**(f) Effect of Movement of Silt and Ice**

Effect of earthquake movements on ice and silt pressures has not been investigated in detail. It is generally believed that increase in pressure due to ice and silt during an earthquake is not of material importance on dams. This is reasonable as the formation of ice is restricted within a few feet of the water surface and the depth of silt touching the upstream face of the dam is generally small.

**(g) Movements on Faults**

Normally, a dam foundation intercepted by active faults should be avoided. Fault movement is not necessarily confined to the fault on which the earthquake originates, but secondary movements may occur on any active fault in the disturbed area. The force with which slippage may occur on a fault is immeasurable. Any prominent fault though apparently dead may be subject to movement in an earthquake. It is extremely difficult to construct earthquake proof dam on a fault.

A typical masonry dam section 130 feet high has been designed to be safe against earthquake, considering seismic forces mentioned in paras (a) and (c) of pages 35 & 36 as other forces are negligible. Computations are attached for easy reference with value of $\alpha = 0.2$. Similar computations were made for heights varying from 20 feet to 130 feet considering value of $\alpha = 0.1$ and 0.2. The design of the profile and position of the middle third points have been represented in Fig. B1 for a non-overflow section and in Fig. B2 for an overflow section showing the extent of extra material required to flare out the profile of the dam near its base.

A separate graph has been drawn (Fig. B3) to show the relation between the height of the dam and the cross-sectional area required.

It will appear from the graph that for a dam of 130 feet height approximately 12% extra concrete will be required, considering $\alpha=0.1$ and 40% extra concrete considering $\alpha=0.2$.

## II. EARTH DAMS

Although damage to earth dams due to seismic effects so far reported is not of serious nature, yet it may be mentioned that in the California earthquake of 1906 an earth dam got ruptured due to the earthquake rift passing directly through the dam. In 1925 in Santa Barbara a similar thing happened. In view of comparatively more seismicity existing in the northern part of India it would be advisable to consider seismic effects in the design of earth dams. An earth dam may fail due to seismic effects as noted under:

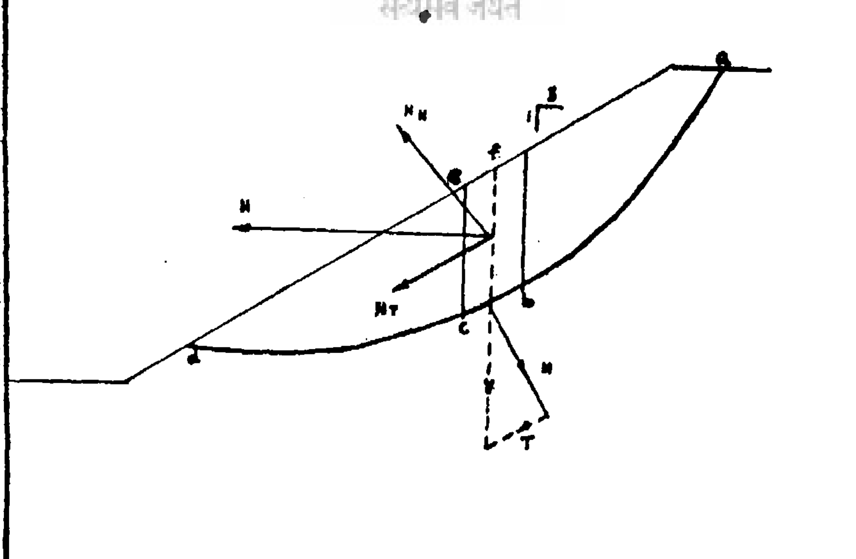
(a) If there be a fault immediately below or near about the area covered by the earth dam.

(b) If an earthquake rift of sufficient intensity passes directly through the dam to cause a rupture in the structure.

(c) If an earthquake vibration in a horizontal direction occurs immediately after a sudden drawdown on the upstream slope of a dam.

For (a) and (b) hardly any precautionary measure can be taken in the design. For (c) the following need be considered.

EFFECT OF EARTHQUAKE IMMEDIATELY AFTER A SUDDEN DRAW DOWN



An earth dam is normally designed to resist failure due to slipping of earth in the upstream slope against a sudden drawdown. When a dam will be subject to horizontal vibration due to earthquake and the structure will be moving in a direction from downstream towards upstream it will have most unfavourable condition so far failure against sudden drawdown is considered. Consider a, b, c, d as the most dangerous circle which offers least resistance to shear in the dam. Let e, f, b, c be a vertical slice of earth bounded by the dangerous circle and the slope of the dam. Resolving the vertical force due to the weight of the earth slice at the circumference of the dangerous circle into tangential and normal directions to the plane b, c we get T=Force tending to produce movement along the circumference of the dangerous circle due to shear.

The force tending to prevent this movement may be represented by N Tan $\theta + lc$ which is equal to the shear strength of the earth material along the small arc b c of the dangerous circle,

where

N=The normal reaction due to weight of the slice.

$\theta$=Angle of internal friction of the material comprising the dam as determined by tri-axial shear test.

l=Length of the arc intercepted by the earth slice

c=Cohesion of the material comprising the dam in lbs. per square foot.

The factor of safety fs may be denoted by

$$\frac{\Sigma N \operatorname{Tan}\Phi + \Sigma l.C}{\Sigma T} = \frac{\Sigma N \operatorname{Tan}\Phi + LC}{\Sigma T} \qquad \text{(i)}$$

Where L=length of the arc a, b, c, d. When the dam will be moving in a direction from downstream to upstream, the earth slice e f b c will be subjetced to an additional horizontal force H which may be resolved into its component forces $H_t$ acting parallel to the tangential force T and in the same direction as T producing movement along the circumference of the dangerous circle and $H_N$ acting parallel to the normal force N, but in a direction opposing it. Considering $H_T$ along with N and T previously determined, the factor of safety f works out to

$$F = \frac{\Sigma N \operatorname{Tan}\phi - \Sigma H\, N \operatorname{Tan}\phi + LC}{\Sigma T + \Sigma H\, T} \qquad \text{(ii)}$$

Comparing equations (i) and (ii), it will be observed that in equation (ii) the numerator has decreased and denominator has increased resulting in decrease in the factor of safety of the dam against sudden drawdown.

To make the earth dam safe against above mentioned seismic forces, it would be necessary to provide a flatter upstream slope than necessary if no earthquake effects are considered. Computations have been made for dams varying in height from 20 feet to 100 feet and the factor of safety against sudden drawdown actually derived have been tabulated and also plotted in Fig. B-4 for ready reference.

**Above results have been based on the following assumptions which are normally met with in Bihar soils.**

| Characteristics | Impervious | Semipervious |
|---|---|---|
| Moist weight in lbs/cft | 112 | 115 |
| Saturated " | 115 | 120 |
| Submerged " | 52.5 | 57.5 |
| Angle of internal friction | 20° | 28° |
| Cohesion in lbs/sft | 1000 | 500 |
| Slope upstream and pervious/zone | --- | 3:1 |
| Slope of impervious zone upstream | 1:5:1 | |

The effect of pore pressure has been allowed as noted under :—

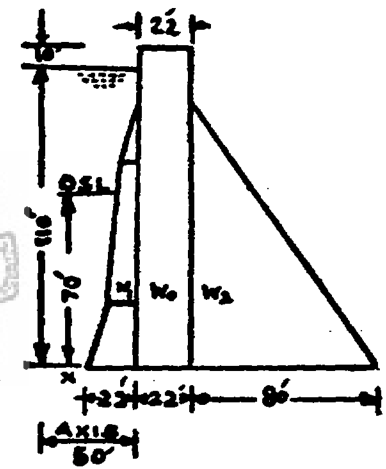
For the portion of impervious zone of embankment, submerged unit weight has been used for calculating normal components and saturated unit weight for calculating tangential components.

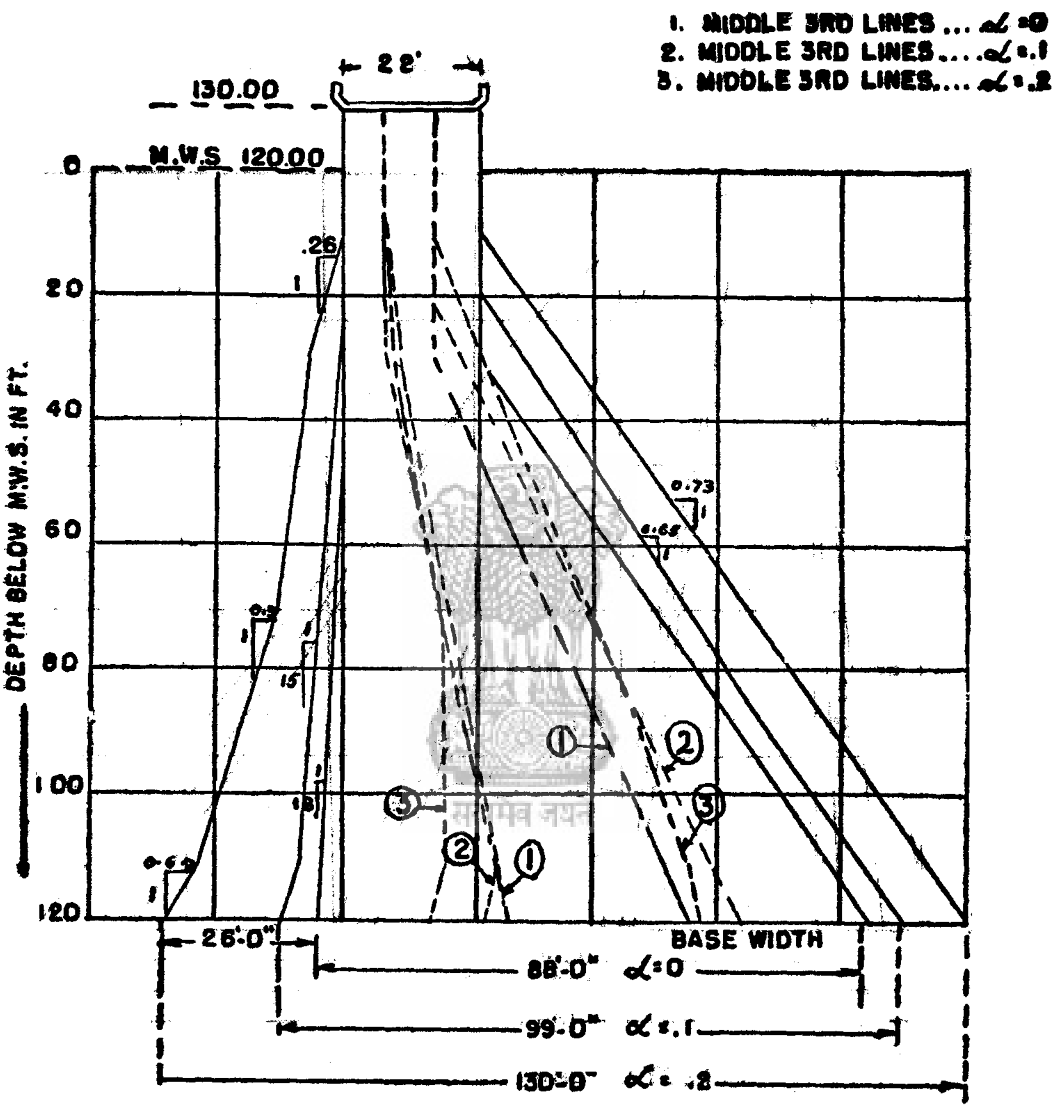
It will appear from the results tabulated that the factors of safety arrived without earthquake effect decrease considerably by considering seismic effects (*vide table below*) :—

| Sl. No. | Height in ft. | Factor of Safety | Factor of Safety with Seismic Acceleration | | | Remarks |
|---|---|---|---|---|---|---|
| | | | 0.1 g | 0.2 g | 0.3 g | |
| 1 | 20 | 4.80 | 3.80 | 3.12 | 2.28 | |
| 2 | 30 | 3.65 | 3.05 | 2.60 | 1.75 | |
| 40 | 2.95 | 2.48 | | | | |
| 4 | 50 | 2.41 | 2.05 | 1.76 | 1.36 | |
| 5 | 60 | 2.28 | 1.89 | 1.65 | 1.32 | |
| 6 | 70 | 2.21 | 1.87 | 1.67 | 1.20 | |
| 7 | 80 | 2.08 | 1.70 | 1.54 | 1.15 | |
| 8 | 90 | 1.95 | 1.66 | 1.43 | 1.12 | |
| 9 | 100 | 1.88 | 1.61 | 1.38 | -- | |

## TYPICAL TRIAL STABILITY COMPUTATIONS FOR SECTION-X

| Symbol | Particulars | H in kips | V in kips | Lever in feet | Moment | Remarks |
|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| | | | | | kip Ft. | |
| Wm | (Weight of masonry) kips | | | | | Distance of Middle third Points |
| | $W_0 = 22 \times 120 \times 0{\cdot}15$ | | 396 | 61 | 24,200 | $Z_1 = 28' + \frac{124'}{3} = 69{\cdot}33'$ |
| | $W_1 = 150 \times$ kips | | 150 | 42·6 | 6,400 | |
| | $W_2 = 80 \times 100 \times \frac{1}{2} \times 0{\cdot}15$ | | 600 | 98·67 | 59,200 | $Z_2 = 28' + \frac{124 \times 2'}{3} = 110{\cdot}66'$ |
| | | | | | 89,800 | |
| Pem | Pressure of Eq on Masonry (Horizontal) | | | | | |
| | $W_{0e} = 396 \times 0{\cdot}2$ | 79·2 | | 60 | 4,760 | When the dam is moving from d/s to u/s |
| | $W_{1e} = 150 \times 0{\cdot}2$ | 30·0 | | 23 | 695 | |
| | $W_{2e} = 600 \times 0{\cdot}2$ | 120·0 | | 33·33 | 4,000 | |
| | | | 1,146 | | 9,455 | |
| | | | | | 89,800—9,455 | |
| | Condition (i) Reservoir empty with earthquake negative | | | (70·00) | = 80,345 | |

| 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|
| Pwh | $\frac{1}{2}\times 1/16\times 110\times 110$ | 378 | | 110/3 | 13,900 |  |
| Pwv | $1860\times 1/16$ | | 117 | 43 (Calculated separately) | 5,020 | |
| P Silt (H) | $\frac{1}{2}\times\frac{67\cdot 6}{1000}\times 70\times 70\times\frac{1}{2}$ | 55 | | 70/3 | 1,280 | |
| P Silt (V) | $640\times\frac{67\cdot 5}{1000}$ | | 43 | 33 (Calculated separately) | 1,420 | |
| Pup lift | $2/3\times\frac{1}{2}\times 124\times 1/16\times 110$ | | −284 | 69·33 (−) | 19,700 | |
| | | | | Total + | 91,701 | |
| | Add Earthquake +ve on masonry | | | + | 9,455 | |
| | Add Earthquake pressure on u/s side | | | + | 3,620 | |
| Condition—Reservoir full with +ve Earthquake | | | | | | |
| | | | Ev 1,028 | (102·3) | 1,04,775 | |



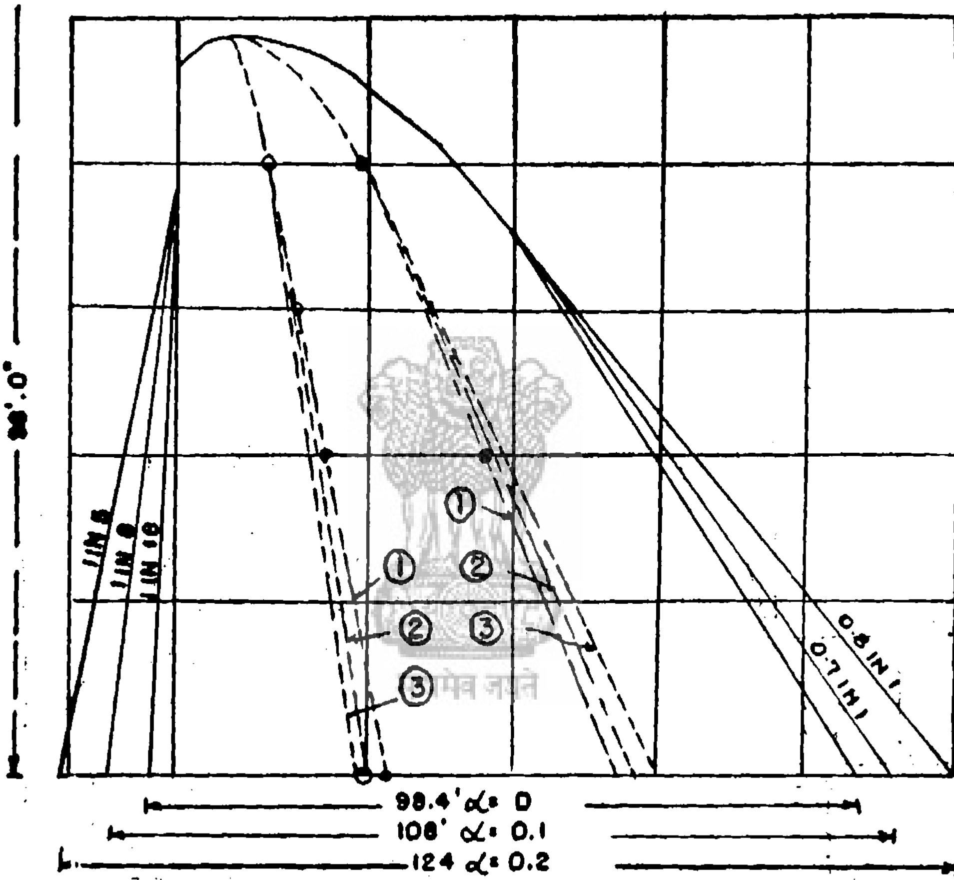

PROFILE OF NON OVERFLOW SECTION TO RESIST SEISMIC EFFECTS

α =0, α=0.15 α 0.2

SCALE 1"= 20'

FIGURE B 1

## PROFILE OF OVER FLOW SECTION WITH $\alpha = 0$, $\alpha = 0.1$ & $\alpha = 0.2$




### EFFECT OF EARTHQUAKE

| CASE NO. | VALUE $\alpha$ | AREA OF SECTION | % INCREASE IN SECTION |
|---|---|---|---|
| 1. | 0 | 6220 SQ FT | — |
| 2. | 0.1 | 6640 SQ FT | 6.0 |
| 3. | 0.2 | 7230 SQ FT | 16.0 |

FIGURE B 2.

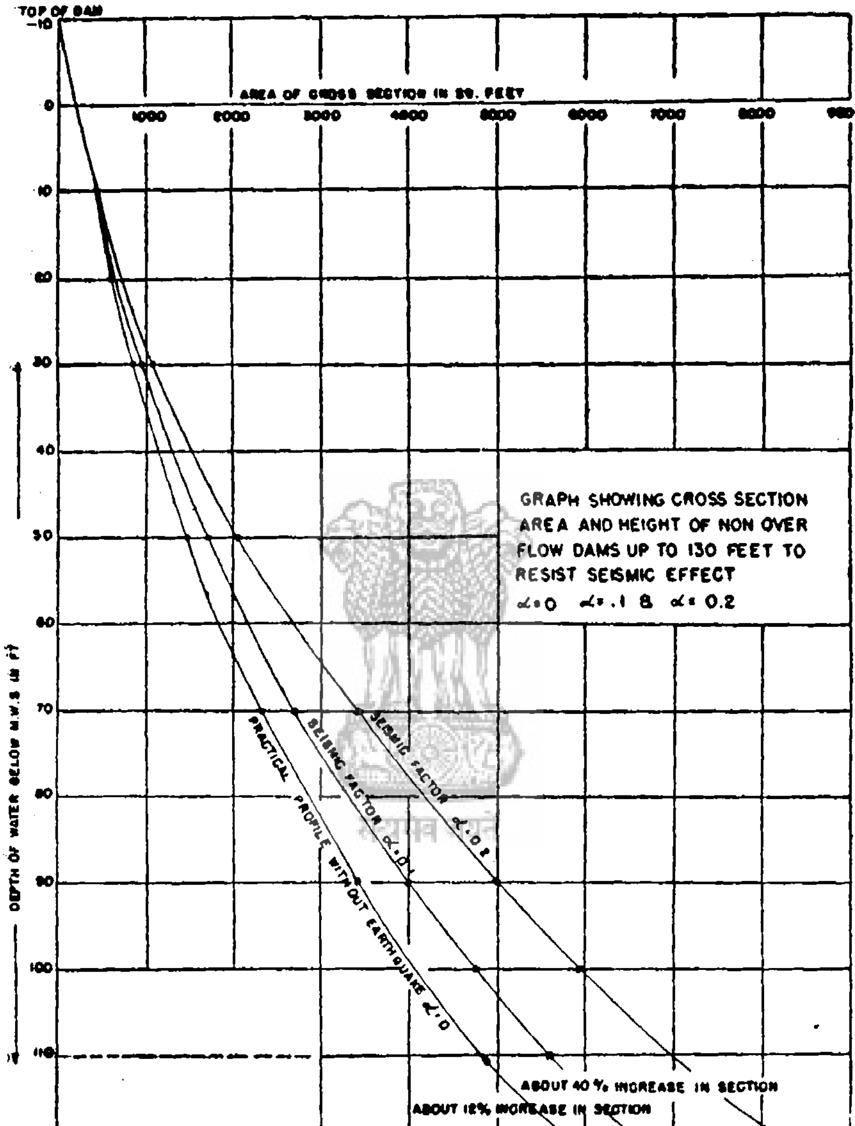

TOP OF DAM
-10
0
10
20
30
40
50
60
70
80
90
100
110
120
AREA OF CROSS SECTION IN SQ. FEET
1000
2000
3000
4000
5000
6000
7000
8000
9000
DEPTH OF WATER BELOW M.W.S. IN FT
GRAPH SHOWING CROSS SECTION AREA AND HEIGHT OF NON OVER FLOW DAMS UP TO 130 FEET TO RESIST SEISMIC EFFECT
α=0 α=.1 & α= 0.2
SEISMIC FACTOR α=0.2
SEISMIC FACTOR α=0.1
PRACTICAL PROFILE WITHOUT EARTHQUAKE α=0
ABOUT 40 % INCREASE IN SECTION
ABOUT 12% INCREASE IN SECTION

FIGURE B 3

FACTOR OF SAFETY AGAINST SUDDEN DRAWN DOWN IN EARTH DAMS OF HEIGHT UP TO HUNDRED FEET

$\alpha = 0.1$ $\alpha = 0.2$ & $\alpha = 0.3$

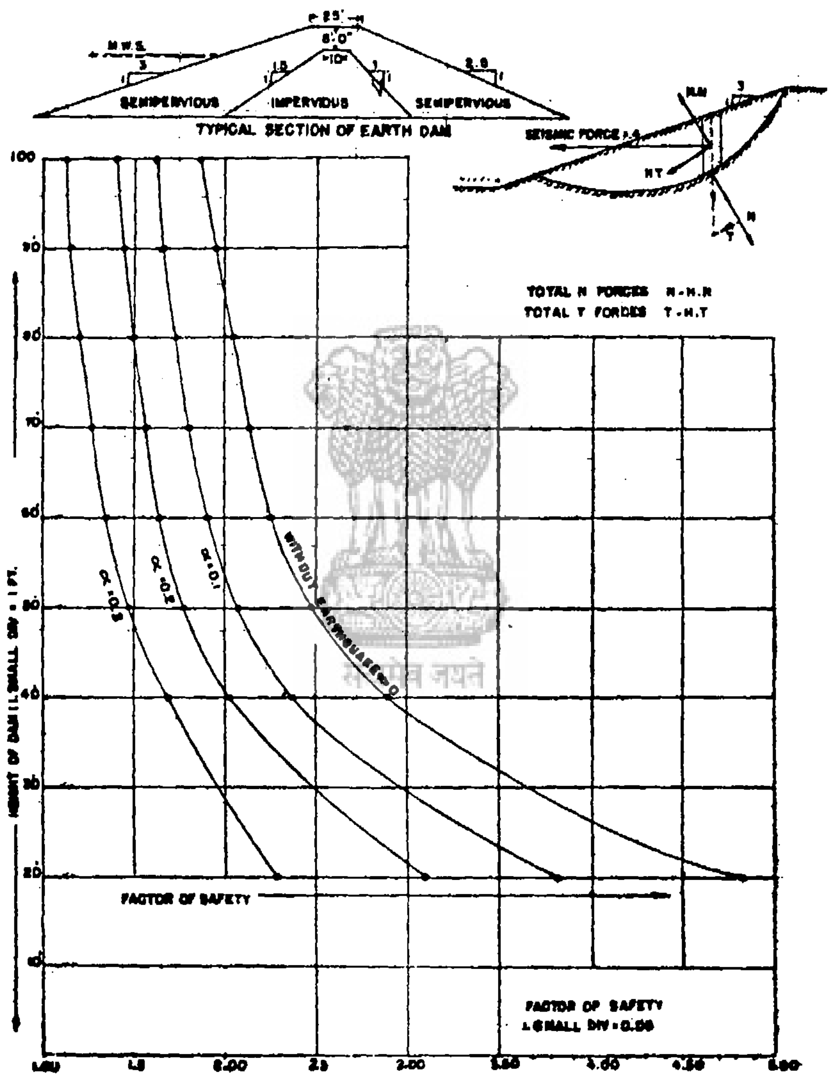



FIGURE B 4

## BIBLIOGRAPHY


1. Westergaard, H. M. "Water pressures on Dams during Earthquakes" Transanctions American Society of Civil Engineers, 1933.

2. Dewell, H. H., "Earthquake resistant construction" Engineering News Record, April 26, 1928, page 650.

3. Lord Rayleigh, "Theory of Sound" Vol. I.

4. Timoshenko, S, "Vibration problems in Engineering".
   Timoshenko, S, "Theory of Elasticity".
   Timoshenko, S & Lessells, "Applied Elasticity".

5. "Earthquake Investigations in California", U.S. Department of Commerce, Coast and Geodetic Survey, 1934-35.

6. Ayre, R. S., "Interconnection of translational and torsional vibrations in buildings" Bull. Seis. Soc. America, April, 1938.
   Ayre, R. S., "Experimental Response of an Asymmetric one story building model to an idealised transient ground motion". Bull. Seis. Soc. America, April, 1943.

7. Fergusson, F. F., "Earthquake Effects on Dams", Central Board of Irrigation Journal, May 1950.

8. Guha. K. & Gurdas Ram, "Vibration of Dams—their free period". Central Board of Irrigation Journal, July, 1950.

9. Greager, Justin and Hinds, "Engineering for Dams" Vols. II and III.

*APPENDIX D*

## DESIGN OF DAMS FOR EARTHQUAKE RESISTANCE*

by

JEROME M. RAPHAEL**

### SUMMARY

Earthquake resistant design of dams differs greatly from the design of the usual type of engineering structures, since earthquake vibration not only generates forces within the structure itself but also increases the magnitude of the external forces which are at all times acting on the structure. Under the action of earthquake, the impounded water behind a dam exerts hydrodynamic as well as the ever present hydrostatic forces on the dam. If failure of the dam were to occur, maximum damage would not be confined to the dam but would be widespread due to the battering ram effect of the suddenly released waters. As a consequence earthquake design for dams tends to be conservative. By their very nature, dams are generally constructed in remote regions where very little engineering experience is recorded. Determination of proper earthquake design factors is the major problem in earthquake resistant design of dams. This is made for a given locality after considering the recorded seismicity of the region and the relative activity of regional geologic processes causing major tectonic movements, such as mountain building. Because of the history of sudden major earthquakes in previously inactive regions, major dams are usually designed for a minimum earthquake acceleration of 0.10 g, ranging up to 0.20 g in regions of maximum seismicity. For a given earthquake acceleration, the hydrodynamic force on the face of the dam depends on the size and configuration of the dam and reservoir. In some cases, choice of type of dams is dictated by consideration of earthquake action. General methods of stress analysis are described for major types of dam.



### EXTRACTS FROM THE PAPER

The next question is : What are the chances of major earthquake at the site? Gutenberg and Richter and Housner have discussed statistical analysis of the shocks originating in a seismic region in a manner which is strongly reminiscent of the analysis of flood probability with which every dam designer is familiar.

Making an analysis of the probability of earthquake of given magnitudes occurring along various seismic belts, by the Housner's method show that an earthquake magnitude 8.2 (Richter scale) (San Francisco, 1906) may be expected in California once every 200 years, while shocks of magnitude 6.25 (Long Beach, 1933) can be expected

*Paper presented at World Conference on Earthquake Engineering held at California in 1956.

**Associate Professor of Civil Engineering, University of California.

somewhere in California about every 18 months. A similar calculation made for the Himalayan arc showed that the expected frequency of occurrence of the Assam earthquake of August 15, 1940 of magnitude 8.4 was once every 500 years, while the expected frequency of occurrence of an earthquake of magnitude 8.3, such as was computed for the Bihar-Nepal earthquake of January 15, 1934 is once each 320 years. Despite statistics, these two earthquakes occurred within 16 years of each other.

The history of many major earthquakes occurring in a region where none were recorded before imposes caution on the design engineer. One method of providing for the contingency of an unexpected shock in a previously quiet region is to adopt a minimum design earthquake of 0.10 g anywhere when considering the safety of a major dam. This follows the recommendations Westergaard made after considering the characteristic vibration of earthquakes, and the normal response of massive gravity dams. His recommendation of an earthquake acceleration of 0.10 g has been accepted in the vast majority of dam designs, except where close proximity to a zone of high magnitude earthquakes has warranted an increase. In designing Bhakra Dam, for instance, which is located to the north of New Delhi, India, designers used a horizontal earthquake acceleration of 0.15 g. Similarly, in the design of a dam proposed for a location 30 miles from the track of the Himalayan arc, accelerations of 0.2 g horizontally and 0.10 g vertically were considered.

## EARTH DAMS

Since earth dams are constructed of materials readily available in the vicinity, designs are varied to fit the properties and quantities of materials on hand. However, there are definite points of similarity among all earth dams, and the following discussion will be of a typical earth dam.



In general, earth dams are characterized by flattened upstream and downstream slopes. Resistance to the passage of water is given by a core of relatively impervious material, buttressed up and downstream by one or two zones of successively more pervious material, which give stability to the structure. The actual materials of the impervious zone is a mixture of solid mineral particles, water and air, the strength of which is a function of the internal friction of the mass, cohesion between particles, and pressure.

Hydrodynamic forces from the reservoir are much reduced in the case of the earth dam because of the flattened slopes. Mass forces in the embankment are tremendous, because of the large volume of materials. The actual values of the mass forces to be used in analyzing a particular cross-section for stability is under discussion at present. Most designers use a seismic coefficient for finding horizontal earthquake forces, which is constant anywhere in the dam. Others, taking into account the dynamic action of the embankment itself, advocate the use of seismic coefficients that increase in some manner from base to crest of dam, much like the increase in seismic coefficients in framed structures from lower to upper floors.

The most common method for evaluating the stability of the slopes is by the Swedish slip circle method. Failure is assumed to occur along the arc of a circle, and forces that cause a mass of earth above the failure arc to move are evaluated against the forces along the arc that resist the movement. The safety factor is expressed as:

$$SF = \frac{C - \tan\theta\,(N-P)}{T}$$

where

C is cohesion,

tan $\theta$ is tangent of the angle of internal friction,

P is pore pressure,

N is summation of the normal forces, and

T is summation of the tangential forces.

The factors in the numerator of the above equation express the shearing resistance of a soil. It has been found that shearing resistance of soil is greater under dynamic loading than under static loadings. Casagrande and Shannon found that the dynamic shearing resistance of clays increased 40 to 160%, sands increased 20% and soft rock increased 80%, over the static resistance. Seed and Lundgren found a 10% increase in shearing resistance of dry sand and even greater increases in saturated sand. Thus as, in many other cases of engineering structures under transient loadings, the increased earthquake forces on an earth dam are to some extent balanced by increased resistance to stress and deformation.

Special provisions in a seismic design of earth dams include:

1. Carrying cut-off trench deep into foundation to intercept water flowing through foundation cracks.
2. Making the impervious section wider than any possible offset.
3. Prevention of slumping by weighting down inpervious section with coarse pervious material.
4. Providing extra heavy outer slopes to close any tension fissures developed by stretching the dam.

Two earth dams in California successfully withstood the 1906 earthquake. Crystal Springs Upper Dam, which had water on both faces, was actually offset by fault movement in the foundation but did not slump under the action. San Andreas Dam, 95 feet high above stream bed, although cracked, developed no leaks and is still used. On the other hand, a 25-foot high dam in southern California slid out on its foundation during an earthquake. With proper engineering design, earth dams can resist seismic forces as well as any other structure.

# SPECIAL PROBLEMS RELATING TO THE CONSTRUCTION OF DAMS IN ACTIVE VOLCANIC COUNTRY

by

J. K. HUNTER AND H. G. KEEFE*

## SUMMARY

The paper defines the special problems affecting the design and constructions of dams and other engineering works which may arise in active volcanic country. It discusses the nature and origin of earthquakes, their measurement and the additional forces to be provided for in engineering structures. A few examples are given of destructive earthquakes which have occurred in the past and suggestions are offered for the accelerations which should be allowed for in design.

A general comparison is made for the relative inherent stability of various types of dam under earthquake shock.

The engineering problems associated with the construction of dams on imperfectly consolidated deposits are discussed and various means of reducing percolation to safe values are described. A comparison is made between alternative methods of constructing watertight cut-offs for embankment dams.

## EXTRACTS FROM THE PAPER

### The Nature of Earthquakes and their Effect on the Design of Dams

The term "earthquake" is applied to the shock waves set up in the earth's outer crust as the result of the sudden readjustment of internal stresses. These stresses arise as a direct result of the thermal changes caused by radio-activity within the earth. The rhythmic character of these thermal changes is believed to account for the successive periods of mountain building and land movements which are apparent from a study of the surface rocks.

Crustal stresses are usually greatest in regions of pronouced instability such as those associated with recent orogenic activity where the weight of the upstanding mountain masses tends to exert a radial thrust. This thrust in turn sets up tangential compressive stresses in the earth's crust which are ultimately relieved either gradually by plastic flow with consequent crumpling and folding of the earth's surface, or by a sudden fracture of the crust, resulting in a raising or lowering of the land along the crack which has developed. Except near their origin, the main direction of these tangential stresses is generally well-defined though varying weaknesses in the crust will result in local deviations. Depending upon the magnitude of the resulting shock, the general effect can be wide-spread or confined to local areas.

*Paper presented at the Fifth International Congress on Large Dams.

In regions liable to earthquakes, relief of crust stresses may occur through a series of small fractures, spread over a period of time and accompanied by small shocks, or it may occur with catastrophic suddenness. Major adjustments, however, are inevitably of the latter type, generally followed by a period of comparative quiescence during which there is a slow and gradual build up of a stress ultimately resulting in another sudden major readjustment. To some extent the existence of a major surface fault will act as a safety valve, since there is a tendency for stress relief to occur gradually and continually at weak spots in the crust.

At the seat of the disturbance (epicentre) the sudden release of the energy stored in the stressed rocks sets up a series of vibrations, the resulting accelerations making themselves manifest in the form of shock waves. These waves are closely harmonic; their amplitudes and periods may vary over a wide range, depending partly on the focal origin of the earthquake, its depth from the surface and its distance from the point of measurement; they can be detected and recorded by a seismograph.

Following the initial shock, a series of further vibrations is almost invariably recorded. These are due to the transmission of the original vibrations either through the earth's core or ground the longer path of the great circle on which the primary wave has travelled.

The focal origin of an earthquake is commonly about 15 km below the surface although very much greater depths have been recorded.

The first intimation of an earthquake is the occurrence of a primary shock wave travelling at 7 to 8 km/s, with an amplitude generally of only a fraction of an inch, although up to 3 inches have been measured in very great earthquakes. This is followed by secondary waves travelling at about half this speed and with amplitudes up to about 4 inches.

The period of the shock wave may vary perhaps from 0.05s to as much as 80s, the principal waves usually having periods between ½ and 2½s. The maximum acceleration imparted to the ground is usually a small fraction of that due to gravity, although in Japan values as high as 0.5 g and 0.43 g were estimated at Kwan-to and at Mino-Owari respectively.

A structure is most vulnerable under repeated shocks of similar amplitude, and experience indicates that such shocks are most likely to occur when the period is between 0.8 and 1.5s (the period for the principal shock in the San Francisco earthquake of 1906 was 1s and that in Tokyo in 1923 was 1.35s). Other periodicities outside the range mentioned appear to damp down rapidly and provide on time for the development of dangerous oscillations.

The San Francisco earthquake of 1906 resulted in a permanent displacement of up to 22 feet on one side of a fault extending over a length of 275 miles, while the Tokyo disaster of 1871 is said to have caused a displacement of 13 feet horizontally and 19 feet vertically.

The greatest vertical displacement so far recorded occurred at Yakutat Bay, Alaska, in 1899 when 47.5 feet was measured.

The area over which an earthquake may be destructive may extend to thousands of square miles. In the Assam earthquake of 1897 the total area affected by the shock was nearly 2 million square miles, the area of heavy destruction extending to over 9,000 square miles.

To obtain full knowledge of the local behaviour of an earthquake at least three seismographs are required, two at right-angles to each other to determine the magnitude and direction of the horizontal shock and third one to determine the vertical or radial shock. By comparing the three sets of records obtained from a number of different stations, the location of he earthquakes focal centre can be closely determined.

Experience suggests that the epicentre of an earthquake rarely, if ever, occurs in a mountain mass itself, but in the adjacent foothills, plains or sea bed. It follows therefore, that dams located in a mountainous region are less liable to suffer damage from an earthquake than those constructed in more level regions, since it is only the effect of a shock transmitted from a distant seat of disturbance against which provision has to be made. On the other hand dams constructed in sub-mountain tracts and especially in the plains are more liable to disastrous foundation movement.

Since a dam may be subjected to a horizontal thrust from any direction, it will be apparent that in the case of a straight gravity dam a thrust along the axis can only be seriously harmful if the earthquake should give rise to faulting or permanent deformation of the ground.

The main threat to the stability of a straight gravity dam arises from a horizontal thrust in a direction at right angles to the axis. This horizontal thrust is due to :

(a) the inertia of the water resting against the dam; and

(b) the inertia of the dam itself relative to the foundation on which it rests.

The effect of (a) depends upon both the slope and the character of the upstream face. A flat upstream slope will reduce the horizontal component of the force due to the shock wave, while the material of which the dam is built will affect its ability to absorb a shock wave without damage. For this reason earth and rock-fill dams are better able to resist earthquake shocks than are solid concrete dams with their predominant vertical faces.

The effect of (b) depends upon the density of the material of the dam and the position of its centre of gravity in relation to the foundations.

The usual method of taking account of the inertia effect of a solid concrete dam is to assume an additional external horizontal force equal to the mass of the dam multiplied by some arbitrary percentage of the gravitational acceleration g. The allowances made in the case of a solid concrete dam have generally ranged from 0.1 to 0.2 g although in some cases an allowance of only 0.05 g has been made.

Experience has shown that accelerations of the order of 0.25 g can occur over quite large areas in the vicinity of the epicentres of severe earthquakes and it is reasonable to suppose that considerably higher values can occur immediately over the seat of disturbance. It is believed, however, that accelerations in excess of about 0.3 g would be accompanied by permanent and severe deformation of the ground surface and that even though the earthquake forces did not immediately rupture the dam itself, foundation or abutment failure would nevertheless occur.

While the empirical method described above is still commonly used, a more advanced technique based on the ground motions likely to be experienced has recently been developed.

In the case of a solid concrete dam of conventional cross-section, it is clear that when the reservoir is empty an acceleration in a downstream direction is the most unfavourable since it is equivalent to a horizontal force in an upstream direction tending to overturn the dam about its heel. It can, however, be demonstrated that on the basis of any normal earthquake acceleration a dam under these conditions would be prefectly stable.

With a full reservoir, however, an upstream acceleration is the most unfavourable because not only does it apply an additional overturning force, but the inertia of the water against the face of the dam imposes a further force in a downstream direction. It should be observed that due to the differences in the elastic properties of the dam structure and the water the impulse forces set up are out of phase with one another and in consequence their total effect is less than their arithmetic summation.

It is generally considered unnecessary in dam design to take an account of a vertical acceleration. In the first place the vertical component of the acceleration set up by the average earthquake will, unless the dam is close to the epicentre, be small compared with the horizontal component. Moreover, the primary effect of such vertical accelerations is merely to vary the apparent specific gravities of the dam and the water without altering their ratio. There is, however, an important secondary effect due, as in the case of horizontal acceleration, to the differences in the elastic properties of the dam and water; the effect is, however, relatively small.

Due to the plasticity of the materials of which they are built, the effect of earthquake acceleration on earth dam is much less serious than in the case of solid concrete dams. Experience indicates that in an adequately designed earthfill dam the inherent factor of safety against slips, both upstream and downstream, is ample to provide against earthquake shock. To a somewhat lesser extent this is also true of rockfill dams.

Precautions which should be taken in the design of an earthfill dam include :—

(a) a liberal freeboard; and

(b) a section largely composed of materials which have self-sealing properties.

With regard to (a) it may be mentioned that during the Second World War the maximum level of all reservoirs in the United Kingdom dependent on earthfill dams was lowered by 5 feet to reduce the risk of overtopping in the event of subsidence as the result of bombing.

Where a core of impermeable material having plastic properties is used, additional thickness should be provided to reduce the risk of leakage following fracture as the result of earthquake shock.

If an interior concrete core-wall is employed special precautions are necessary to ensure flexibility at the plane of junction between the dam and its foundations, while strong reinforcement throughout the height of the core-wall is essential.

The use of a thin concrete membrane on the upstream face is undesirable since even if heavily reinforced it is liable to be displaced by earthquake shocks, especially if these should be in a longitudinal direction.

In general it may be said that a well-designed earthfill dam is the safest of all dams under earthquake conditions. This applies specially to dams on a rock base, but where the ground is soft there is a danger of greater settlement.

The determination of the best type of earthquake resistant design in any given case depends mainly on three site conditions, namely; (1) type of foundation, (2) probable direction of seismic shok (derived from past observation); and (3) materials available for construction.

Summarising, the most suitable types, in order of their inherent resistance to earthquake shock, are as follows:

Earthfill.
Rockfill.
Solid concrete gravity.

All these types can be made to withstand earthquake shocks provided the latter are not of such a magnitude as to cause permanent rupture or deformation of the foundations. Should such deformation occur the dam will fail irrespective of its inherent strength.

In Table 1 are given particulars of three dams in the San Andreas Valley, California, and one in Chile, which have survived severe earthquakes.

TABLE 1

| Name | Location | Date of construction | Type | Height in feet | Remarks |
|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 |
| *San Andreas | San Andreas Valley (California) | 1870 | Earthfill | 95 | Cut-off 35 ft. deep. Raised in 1875 and again in 1928. In the 1906 earthquake the dam moved bodily 8 ft. |

| 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|
| Upper Crystal Springs | San Andreas Valley (California) | 1877 | Earth-fill | 85 | Cut-off 105 ft. deep. Sited across a fault. In the 1906 earthquake it developed several shear cracks along the fault. One part of the dam moved 8 ft. Further cracks developed parallel to its axis. |
| San Mateo (Crystal Springs) | San Andreas Valley (California) | 1889 | Concrete gravity | 154 | Axis of dam parallel to a fault. Built on argillocal careous limestore. Undamaged in the 1906 earthquake. |
| Cogoti | Chile | 1938 | Rock-fill | 246 | Little damaged by severe earthquake in 1943. |

The Cogoti dam in Chile is of special interest since it was specially designed for service in a seismic region and was severely tested in the earthquake of 1943, only five years after completion. It is a rockfill dam, 246 feet high, the design of which was based on the following principles :

(1) Selection of side slopes which would not flatten under earthquake shock.

(2) The use of a water-tight upstream face which could bend without rupture.

The side slopes were chosen after study of natural rock slides which had previously been subject to earthquakes. After dumping the main fill of the dam to its natural angle of repose additional rock was added to flatten the upstream slope which was then finished off with a surface fill of screened and washed gravel, bringing the final face to a slope of 1 to 1.6. On the finished face of the rockfill was laid a laminated reinforced concrete slab designed to ensure maximum flexibility. The downstream slope of the dam was made 1 to 1.8.

It is reported that the 1943 earthquake (which probably reached an intensity of at least 0.2 g) caused very little damage to the dam, the leakage being almost unchanged, while the settlement of the fill which had totalled about 13½ inches between 1938 and 1943 was only increased by a further 15 inches. It is reported that subsequent settlement of the dam has been negligible.

Where tunnels are constructed in rock, earthquake damage is usually confined to the portals. In this connection experience obtained in the Hawkes Bay earthquake in the North Island of New Zealand is of interest. This earthquake which occurred in 1931 resulted in a permanent uplift of 8 feet near the epicentre and of 6 feet at the town of Napier, 10 miles distant, the horizontal acceleration over a considerable area being computed at not less than 0.25 g.

At Tuai, some 40 miles from the epicentre, a 12 feet diameter tunnel, 790 feet long, although driven through very broken country was undamaged, while railway tunnels near Napier were only damaged at the portals or blocked by landslides.

Japanese experience seems to confirm the relative immunity of tunnels from the effect of earthquakes, and this immunity represents an argument in favour of the adoption, where practicable, of an underground arrangement in the case of hydroelectric works constructed in seismic regions.



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*APPENDIX F*

# MEASUREMENTS OF SETTLEMENTS AT CERTAIN DAMS ON THE TVA SYSTEM AND ASSUMPTIONS FOR EARTHQUAKE LOADINGS FOR DAMS IN THE TVA AREA

by

C. E. BLEE & A. A. MEYER*

## SUMMARY

The authors present observations and measurements on the structural behaviour of several dams built by the Tennessee Valley Authority. They also discuss the studies made and the loading assumptions adopted to compensate for seismic forces which might be experienced in certain portions of the Tennessee Valley area.

* * *

The authors outline the studies of seismic occurrences in different parts of the Tennessee Valley, the sub-division of the Valley which was adopted for seismic purposes, and finally they explain the basic assumptions adopted for earthquake loadings. Seismic forces acting on retaining walls are specifically discussed. Model tests on the shaking table to probe the magnifications of the amplitude of seismic motion in fill materials led to the application of an inertia force to the Coulomb wedge, which resulted in an increase of about 35% in the volume of retaining walls.

## EXTRACTS FROM THE PAPER ASSUMPTIONS FOR EARTHQUAKE LOADINGS FOR DAMS IN THE TVA AREA

Of the 20 dams built by TVA, 19 are in the regions where a seismic factor could be omitted. Only one hydro project, Kentucky Dam, is located in the seismic "active" area on the lower river.

Kentucky Dam is a barrier 8,422 feet long, designed to withstand a maximum waterhead of 78 feet and having a maximum height between foundation and deck of 206 feet. The barrier includes three separate earth embankments having a combined length of 6,612 feet. The remaining 1,810 linear feet of the dam is formed by concrete gravity structures, functioning as spillway, powerhouse and navigation lock, respectively.

From the outset it was agreed to apply seismic loads only to those parts of the structure which act as a dam. Structures removed from the dam or superimposed on the dam but not retaining reservoir water in a direct or indirect way were excluded from earthquake design.

*Paper presented at the Fifth International Congress on Large Dams, Paris, 1955.

Seismic loads were not applied to the rolled-fill earth embankments, but the embankments were designed with a wide crest and conservative factors of safety against sliding were used in the analysis for static loads. These factors of safety were 1.5 or better throughout.

A layer of gravel and rockfill with a minimum thickness of 4 feet was placed on all upstream slopes of the embankments.

After considerable study and discussion, the earthquake acceleration applicable to the design of rigid structures founded on rock was taken as 0.05 gravity, with this acceleration acting either parallel or normal to the axis of the dam.

For concrete gravity structures of great rigidity and founded on rock, the loads were increased in the conventional way by adding inertia and water hammer loads to the static loads. The water hammer loads were applied in accordance with Westergaard's theory. For less rigid structures, in which foundation motion may be substantially amplified, an equivalent calculated inertia force up to 0.10g was applied. The application was based upon rational calculations of the dynamic action of the structure.

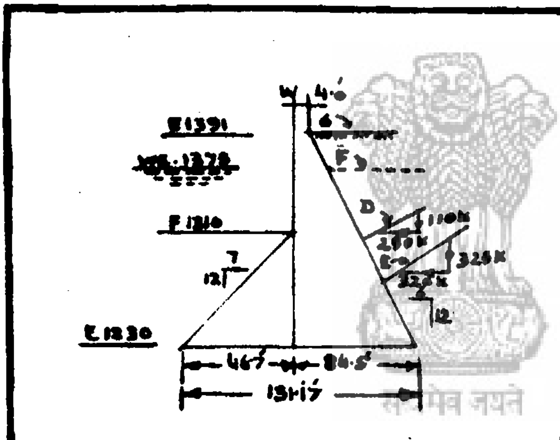
Since the layout called for three separate sections of earth embankment, there were four junctions between earth and concrete structures to be designed. It was at these junction points that the factor of earthquake loading became of real importance, requiring special considerations in the design of the retaining structures.

It is well known that ground motion introduced by an acceleration in rock will be magnified in a fill overlying the rock. Depending on the size and shape of the embankment and the character of the fill material, the amplitude of the ground motion may be increased up to ten or more times the amplitude of the motion in the rock where the vibration originates. Observations on the very severe destructions of docks at Yokohama during the 1923, Tokyo earthquake had led to the investigation of this phenomenon. Mononobe and Matsuo developed a theory as to the behaviour of earth pressure intensities during earthquakes. To clarify the design procedure on the Kentucky retaining walls, TVA initiated shaking table tests which were carried out by Professor Jacobsen at Stanford University, California. The test results were in substantial agreement with Mononobe's theory. Tests were also run to measure water pressures on walls during earthquakes. The result of these tests gave a very close confirmation of Westergaard's theory of water hammer.

Designing in accordance with the results of these tests, an inertia, force of 0.18g was applied to the Coulomb wedge in the manner of Mononobe. This increment was applied as a separate and additional load, with the point of application about 0.60, the height above the base of the wall. Applying seismic loads resulted in an increase of approximately 35% in the volume of the retaining walls. For comparison, the corresponding increase for a concrete dam structure when adding inertia and water hammer loads, with the intensities used at Kentucky Dam, amounts to only 7%. Figure 3b shows a cross section through the retaining wall between the spillway and west embankment with the static earth pressure loads and the seismic load increment indicated. The tabulation of the figure gives the results of stability analysis of the section; the effect of the seismic increment is shown by the difference in stresses between cases II and III.

| Case | $\Sigma V$ Kips | $\Sigma H$ Kips | Stresses lb/sq in fr | fe | fs | Q |
|---|---|---|---|---|---|---|
| I | 1350 | 0 | 71 | 72 | 0 | |
| IA | 1917 | 617 | 127 | 76 | 33 | 102 |
| II | 1558 | 380 | 96 | 69 | 20 | 15.7 |
| III | 1668 | 776 | 178 | 0 | 41 | 7.8 |

Note : Loads are for 1-foot length of wall.




F g. 3 (b)

DESIGN CASES

Case I Dead load concrete structure only

Case IA Dead load concrete plus compacted dry fill

Case II Water surface El 375 saturated fill

Case III Same as case II plus earthquake acting from embankment side

SYMBOLS

D. Dynamic increment.

E. Static earth load.

G. Ground surface.

S. Saturation line.

W. Maximum water surface

$\Sigma V$. Summation of vertical loads

$\Sigma H$. Summation of horizontal loads

fr. Base stress at river side

fe. Base stress at embankment side

fs. Average shear stress on base

Q. Shear friction factor of safety $= \dfrac{0.250A + 0.80\Sigma V}{\Sigma H}$

A. Base area in sq. inches.

Fig. 3b.

Kentucky Dam. Earthquake design of retaining wall Barrage de Kentucky. Calcul sismique du mur de retenue.

*APPENDIX G*

# HYDRODYNAMIC PRESSURES ON DAMS DUE TO HORIZONTAL EARTHQUAKE EFFECTS ENGINEERING MONOGRAPH No. II OF USBR

by

C. N. Zangar

## SUMMARY

This monograph presents a rapid, inexpensive, and accurate method for determining the increase in water pressure on dams, due to horizontal earthquakes and gives the magnitude of these pressures for a number of cases. Although earlier papers have shown that the increase in water pressure on dams due to earthquake is not excessively large, it is an important factor in their design. It has been recognized that water pressures due to earthquake diminish with decrease in the upstream slope of a dam, but data do not exist giving these pressures as a function of slope. Mathematical methods may be used to compute these pressures, but they are complicated and time consuming.

If water is assumed to be incompressible, an electric analog may be used to determine the magnitude and distribution of the water pressure increases caused by a horizontal earthquake on a dam on any profile. Although this assumption is not conservative, a comparison with Westergaard's analytical results for dams with vertical upstream faces shows that for dams under 400 feet in height the error is exceedingly small, and that it is not excessive for dams as high as 800 feet.

The electric analog method consists of constructing a tray geometrically similar to the dam and reservoir area. A linearly varying electric potential is placed along the boundary representing the upstream face of the dam, and a constant electric potential placed along the boundary representing the bottom of the reservoir. The tray is then filled with an electrolyte and the streamlines are surveyed by means of a modified Wheatstone bridge. The distribution and magnitude of pressures on the face of the dam are obtained from the equipotential lines that are constructed from the streamlines.

The increase in water pressure, Pe, caused by an earthquake is given by the equation.

$$Pe = C \alpha w h \qquad \text{(1)}$$

Where w is the unit weight of water, h the depth of the reservoir at the section being studied, and $\alpha$ the horizontal earthquake intensity. C, the unknown quantity, defines the magnitude and distribution of pressures which are determined by the equi-potential lines in the flow net. C is a function of the shape of the dam and reservoir and is unaffected by the intensity of the quake.

The designer need only select a reasonable value for $\propto$ and use the proper C values given herein to determine the water pressures on any dam due to a horizontal earthquake. With the water pressures known the stresses in the dam can be computed by statical methods.

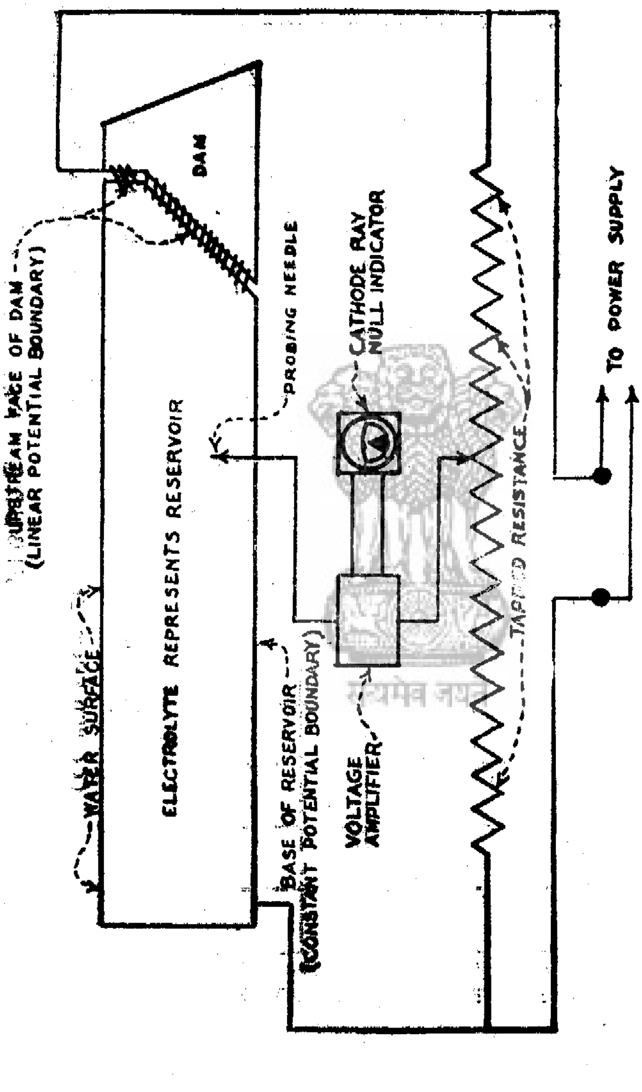
## EXTRACT FROM THE PAPER

**Earthquake Intensities**

In order to determine the total horizontal force due to an earthquake, it is necessary to know the acceleration of the quake or the earthquake intensity. The use of earthquake spectra derived from recorded accelerographs is suggested for determining the intensity. Biot's proposed standard spectrum may be used if a damage scale is applied since it does not include damping. A joint committee of the ASCE and Structural Engineers Association of California has applied a damage factor to Biot's proposed spectrum and has suggested that the maximum value of $\propto$ be 0.10 and the minimum value 0.03 for other than frame structures. The Bureau of Reclamation has consistently used a horizontal intensity, $\propto$ , 0.10 on dams, alongwith a vertical intensity of about equal or smaller magnitude Kosi Dam and Bhakra Dam in India, however, were analysed for a horizontal earthquake intensity of 0.15.

Resonance in dams is not apt to occur for several reasons. The fundamental period of vibration of the usual concrete or earth gravity dam will be from 0.08 to about 1.00 second while the maximum energy of the earthquake appears in most spectra at a period of approximately 0.2 second. Resonance with the foundation is not apt to occur since studies of the fundamental ground periods show values of 0.03 to 0.05 seconds. Although earthquakes are experimentally and analytically treated as harmonic, recorded ground motions do not appear to be harmonic in the destructive zone of the quake, and a steady state response of the structures is usually not established. Also, many forms of damping that are difficult to evaluate act to prevent resonance.

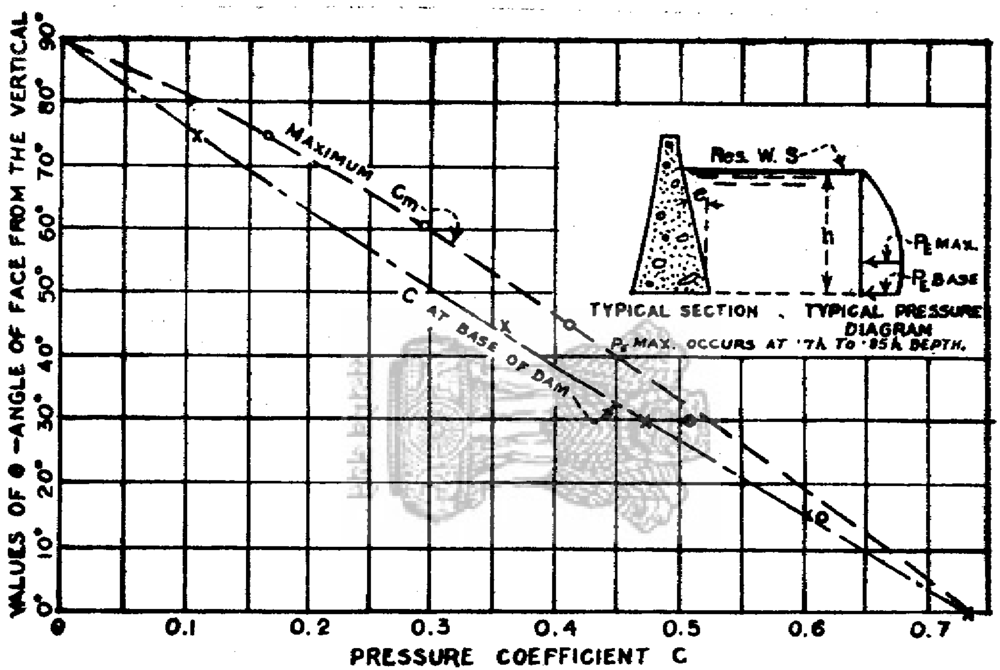
At the present time, the choice of earthquake intensity to apply to a structure must be based upon experience in conjunction with available seismic records. Earthquake spectra including the effects of damping need to be determined for structures having a wide range of fundamental periods. The spectra should be obtained by subjecting the structure with damping to actual recorded accelerograms of destructive earthquakes such as the Helena, Montana quake of 1935, the Ferndale, California quake of 1938, and the El Centro, California quake of 1940.




DIAGRAMMATIC LAYOUT OF ELECTRIC ANALOGY TRAY

(COPIED FROM ENGINEERING MONOGRAPH NO 11, OF U.S.B.R.)

FIGURE F1




PRESSURE COEFFICIENTS FOR CONSTANT SLOPING FACES

(COPIED FROM ENGINEERING MONOGRAPH No. 11, OF U.S.B.R.)

FIGURE F2

*APPENDIX H*

**UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION: COMMISSIONER'S OFFICE**

Building 53, Denver Federal Centre, Denver 25, COLORADO

In reply
Refer to: D-209 F

Dated October 14, 1959.

Airmail

Mr. R. C. Shenoy,
Superintending Engineer,
Assam Investigation Circle,
Central Water & Power Commission,
Ulubari, Gauhati, Assam (India).

Dear Mr. Shenoy,

The following information on earthquake design of Trinity Dam is furnished in response to your letter of September 25, 1959.

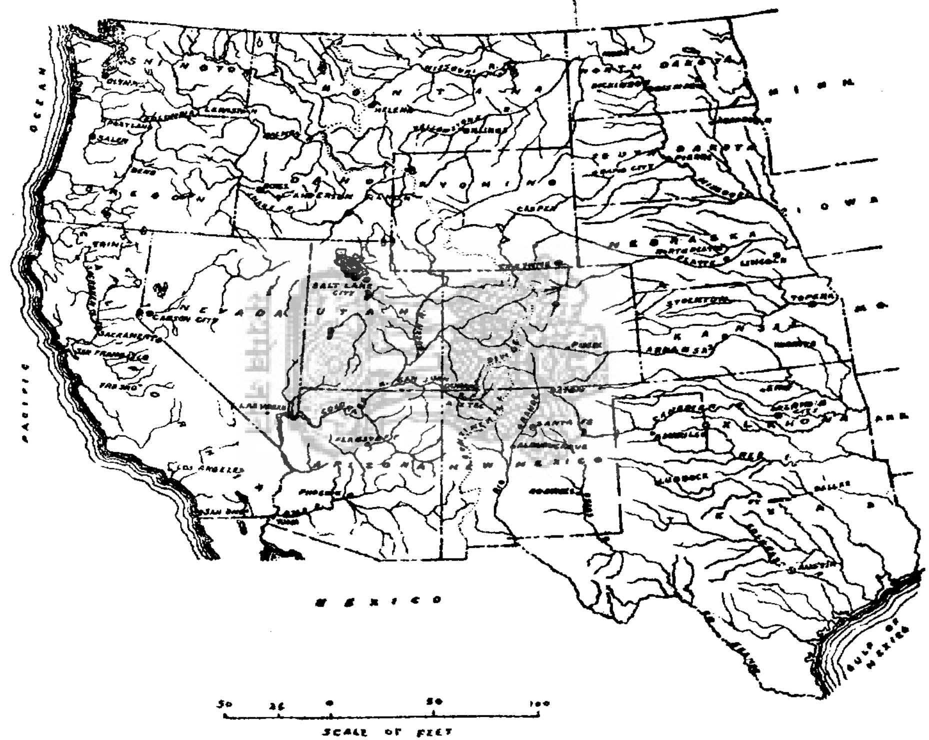
No major faults have been located near the Trinity Dam site. In the design of the dam an investigation of the seismicity of the general area was made. Nineteen recorded earthquakes were found to be applicable. These were analysed and a design value of 0.05g, applied simultaneously vertically and horizontally, was chosen. This value was added to the forces in the conventional analyses for the construction condition, high level reservoir steady state condition and sudden drawdown.

We do not have data on the performance of high earth dams subjected to earthquake. Six years ago we installed accelerographs at a dam in a highly seismic area in Southern California, which eventually will provide information on this important subject.

Sincerely yours,

Sd/- Stephen H. Poe, *Chief*
*Technical Information Branch.*

MAP SHOWING ANDERSON RANCH & TRINITY DAM
U. S. B. R.

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BY U.S.B.R

FIGURE 61

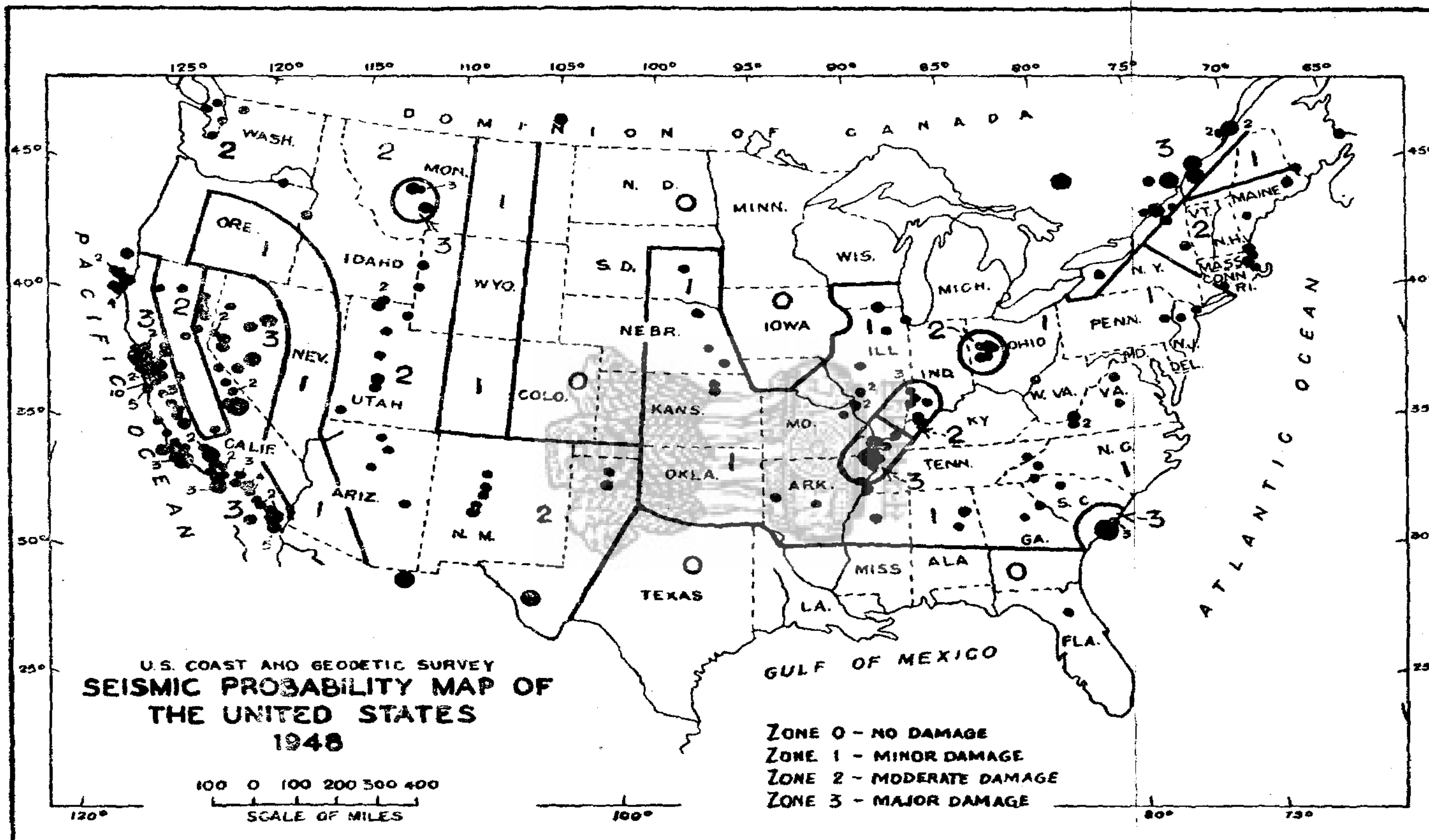


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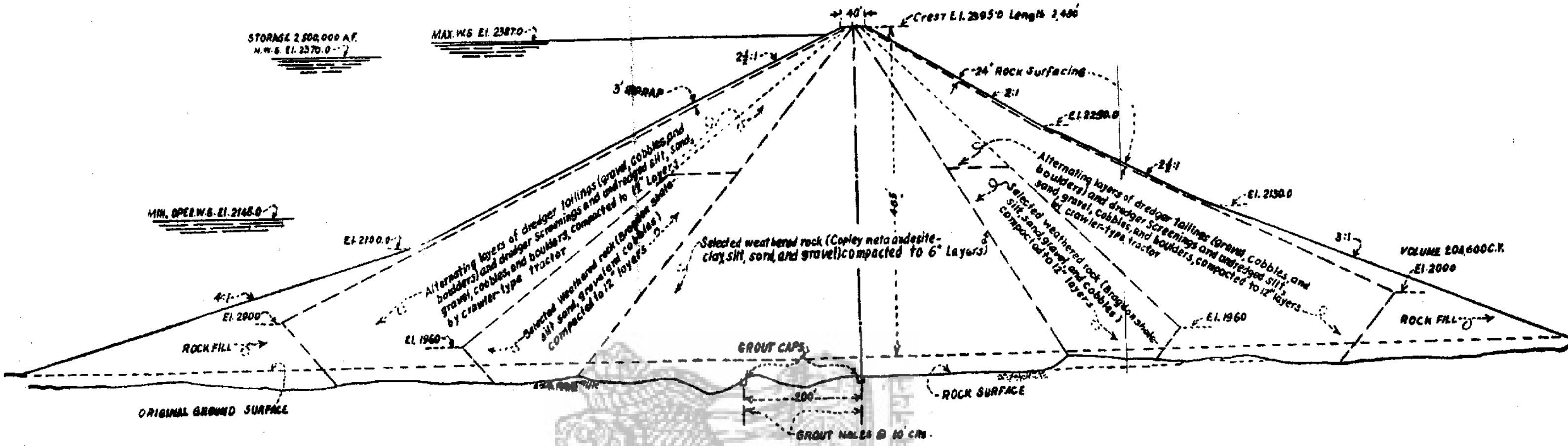
FIGURE 82




VOLUME - 29,129,800 C.Y.

**TRINITY DAM 1957**

CENTRAL VALLEY PROJECT - CALIFORNIA

40 20 0 40 80

SCALE OF FEET.

COPIED FROM
"DEVELOPMENT OF EARTH DAM DESIGN"
BY
U.S.B.R

सत्यमेव जयते

*APPENDIX I*

Rome, January 21, 1960.

Mr. K. L. Rao,
Government of India.
Central Water & Power Commission,
New Delhi, India.

My dear Mr. Rao,

I reply to your letter of December 19, 1959, asking me for news concerning the building of dams in seismic areas.

Regarding this matter, no specific rules have as yet been issued in Italy which regulate the planning and construction of dams in declared seismic areas, however, the problem has always been vital even if solved with arbitrary solutions based on statistical elements regarding, above all, the static behaviour of works that sustained intensity seismic action tests.

Following the idea of adopting structures that can react elastically to dynamic actions, the arch and gravity-arch types have always found major credit, for mural structures, provided that the abutment soil offers a good homogeneity both as regards the mechanical features as well as the nature of the earth's soil.

The multiple arch type structure, even if Italian regulations do not exclude it, was not adopted for a general conviction, both with the planners and the controlling channels, that such a structure, with its elements constituted by arches and jetties, cannot react homogeneously to dynamic actions.

The gravity type, like the more modern one with jetties, favourable geognositc conditions of the nature of the earth's soil recurring have been frequently adopted even in declared seismic areas.

Other cases concern solutions relative to loose materials dams; earth and dry masonry, considered comforting, especially when, in the latter the stress element consists of a nucleous of argillaceous material easily deformable as it occurs for the earth dams.

Briefly, it can be stated that earthquakes, even of important intensity, which occurred in Italy in the present century, have not caused any damage to the existing storage dams located in the areas struck by seisms and conforming to the above mentioned ideas.

For information purposes I may add that recently the problem of seismic action on dams was the object of experimental trials carried out by the I.S.M.E.S. of Bergamo, under the direction of Prof. Guido Oberti, on the reduced-scale model of the Ambiesta Dam, of the double curvature single arch type, of the Societa Ardiatica di Elettricita, planned by Eng. Carlo Semenza.

The work, subdued to very severe seismic actions, has given excellent results showing excellent degrees of resistance and stability with up-heaving and undulative seisms of exceptional intensity. However, the corresponding geognostic conditions of the abutments soil, faithfully reproduced on the model, were not at all defined as favourable.

As a rule, the building of mural works in areas declared seismical is only allowed when the soil offers excellent mechanical features, sufficiently uniform, or when any eventual deficiencies result definitely restorable with appropriate omogeneizing works.

In the caculation of stability it is advisable to bear in mind, in addition of the action of its own weight and that of the water, seismic actions, compared in simplified form :

—for the masonry inertia to an increase or a decrease of its own weight not below 20% for upheaving seisms and 10% for undulative seisms, acting horizontally in each direction;

—for the water inertia, said comparison is advisable for a uniformly distributed pressure on the face wall not less than 5% of the maximum hydrostatic pressure.

For second class seismic areas, said additional forces can be reduced to one-half.

Said criteria, to be accepted provisionally, seem sufficiently prudent. In this connection I recommend that the following memorandum be consulted :

"Le azioni sismiche sulle dighe"
(Seismic actions on dams)
by Francesco Penta and Giulio Supino.
Giornale del Genio Civile
(Civil Engineering Bulletin)
No. 7-8, July-August 1957, page 499.

To date, in our country, no collapse of earth or rock dams occurred.

Sincerely yours,

Sd/- M. Visentini

## THF EFFECTS OF SEISMS ON DAMS

### SUMMARY

(1) Justifies the formulation of article 25 of the regulations for dams. (2) Some preliminary remarks relating to seismic phenomena are given. (3) The exact measuring of seismic action is explained. (4) The most dangerous period of vibration of earthquake shocks. (5) Effects of an earthquake shock from a geological-technical point of view. (6) Special studies on dams made by Italian and foreign investigators. (7) Accelerations produced by earthquakes. (8) The foundation rock as a fundamental factor which determines the resistance of a structure. (9) Technical regulations in force for calculating the resistance of structures built in seismic regions. (10) Criteria for the design of dams given by various authors. (11) Theory of Westergaard. (12) Stability of dams in the presence of seismic actions according to the authors of this paper. (13) An example of the variation of thickness of a massive gravity dam with triangular profile, calculated on the basis of the anti-seismic rules. (14) Criteria for estimating the degree of seismicity of a given locality. (15) Conclusions.

### ANNEXURE

#### Text of article 25

#### as approved by the Superior Council

1. The first Italian regulations for the design, construction and maintenance of barrage dams was dated 1925 (Royal Decree No. 2540, dated 31-12-1925). The regulations were later brought up-to-date in 1931 (Royal Decree No. 1370, dated 1-10-1931).

Recently the Ministry of Public Works decided that a new revision of the regulations was necessary. So by an inter-ministerial decree dated 1-12-1954 (No. 18569), a technical commission was appointed for the modification of the regulations. This commission concluded its work in October 1956.

The text of these regulations has been approved by the respective Superior Council. The new text differs in some parts from the two previous documents. Among other points we may mention that a new article (No. 25) appears in the new regulations for the first time, prescribing special precautions for the construction of masonry dams in earthquake zones.

The drafting of the different articles of the regulations was obviously the collective work of the commission. However, we think we may state that the prescriptions given in article 25 were mainly due to our suggestions. As it is to be expected that this article may give rise to controversies, we wish to give briefly in this paper the fundamental technical reasons that led to the formulation of these prescriptions.

2. We shall begin with a few observations on seismic phenomena.

Earthquakes are due to various causes, e.g., they may be originated by the collapse of caverns in the subsoil layers, by volcanic phenomena, by fractures; in these three cases the earthquakes are generally of slight intensity and normally of local character. Or they may be "fault earthquakes", due to the overcoming of the limiting friction between the two faces of the fault; in this case the earthquakes originated are of great intensity.

Earthquakes which originate at a great depth and are due to causes which are not yet fully known are of less frequent occurrence.

In view of the settling down which is taking place at present in different parts of the earth's surface, there are zones which are subject to earthquakes with a certain degree of frequency. Such areas are called seismic zones.

We should at the outset draw attention to the fact that the intensity of an earthquake is not solely related to the intensity of the original shock. Different geological factors also exert a direct influence on the intensity. The principal influences are nature, strength and degree of external alteration of the topmost superficial geological formations, their stratigraphy and tectonics in relation to the morphology of the ground.

These factors contribute, in different measures and in different directions, to the increase or the decrease of the shocks registered in a given locality. When we know these factors, it is possible to estimate beforehand the maximum seismic effects on the surface of the given locality*. We shall return to this point later.

3. In what does a seismic action consist? An earthquake is a sudden and rapid succession of vibrations of the earth's surface, either in the vertical direction (upheaval earthquakes), or in the horizontal direction (undulatory earthquakes). The two expressions "upheaval shock" or "undulatory shock" indicate, however, only the predominating nature of the vibrations. In reality a point in the zone traversed by an earthquake describes a closed trajectory, which is generally nonplanar and very variable in direction. Such a trajectory may be seen in Fig. 1, which represents the projection on a plane of the actual trajectory of a point.

The disturbed locality, which may be at a greater or smaller depth, or at most on the superficial crust, where earthquake shocks originate is called the hypocentral zone. If the zone is at a great depth, the hypocentral zone may be represented by a point (hypocentre). The name epicentral zone is given to the surface area which experiences the most intense effects of a given earthquake; most generally, the epicentral zone lies vertically above the hypocentral zone. Around the epicentral area, if the number of observations is sufficient we can trace he iso-seismal curves (which are the curves joining the points on the surface where the shock was registered with the same intensity). Similarly we can trace the homo-seismal curves,

* F. Penta : Notes on lessons in Technical Geology, Napoli, 1951, page 35.

which are the curves passing through the points of the surface where the shock was registered at the same instant. These curves are very irregular, for the reasons already mentioned in section 2 above, and those which we shall mention below in section 5.

Earthquakes, as felt in a given locality, are classified, depending on the effects produced, into 12 different degrees of an empirical scale called the Mercalli scale.

The degrees of the Mercalli-De Rossi-Forel scale, as completed by G. B. Rizzo (Lessons in Geophysics 1929-30), who added the last two degrees, are as follows:—

*Degree I*

Instrumental earthquake (registered only by seismographs).

*Degree II*

Very light earthquake (felt by few persons in repose).

*Degree III*

Light earthquake (felt by persons in repose, who are moreover able to estimate its duration and direction).

*Degree IV*

Moderate earthquake.

*Degree V*

Intense earthquake (generally felt by all the people in the locality).

*Degree VI*

Very intense earthquake (felt by everyone indoors, and by some of them with alarm).

*Degree VII*

Most intense earthquake (felt with alarm by most people indoors, some rushing outside the houses and accompained by the fall of chimneys and tiles and injury to buildings).

*Degree VIII*

Disastrous earthquake (causing great alarm to the population, partial destruction of some houses and considerable injury to other houses).

*Degree IX*

Calamitous earthquake (total destruction of some houses and serious injury to many others).

*Degree X*

Very calamitous earthquake (destruction of many buildings even if solidly built, slight changes in the configuration of the ground).

*Degree XI*

Catastrophic earthquake (total destruction of cities, sinking or sliding of mountains and marked changes in the configuration of the ground).

*Degree XII*

Great catastrophe (total razing down of cities, opening of chasms in the ground, etc.).

4. It is possible to find from any seismogram (even if it is irregular) the maximum amplitude of vibration and the maximum acceleration occurring in the surface layer of the earth. Cancani correlated the acceleration with the degrees of the Mercalli scale and obtained the following results :—

| Degree | | Maximum acceleration | | | |
|---|---|---|---|---|---|
| I | From | 0 | To | 0.0025 | m/sec² |
| II | ,, | 0.0025 | ,, | 0.005 | ,, |
| III | ,, | 0.005 | ,, | 0.01 | ,, |
| IV | ,, | 0.01 | ,, | 0.025 | ,, |
| V | ,, | 0.025 | ,, | 0.05 | ,, |
| VI | ,, | 0.05 | ,, | 0.1 | ,, |
| VII | ,, | 0.1 | ,, | 0.25 | ,, |
| VIII | ,, | 0.25 | ,, | 0.50 | ,, |
| IX | ,, | 0.50 | ,, | 1.00 | ,, |
| X | ,, | 1.00 | ,, | 2.50 | ,, |
| XI | ,, | 2.50 | ,, | 5.00 | ,, |
| XII | Beyond | 5.00 | m/sec² | | |

A comparison of other earthquake intensity scales with the Meralli scale may be found in the various treatises*.

The Messina earthquake of 1908 and the San Francisco earthquake of 1906 probably gave rise to maximum accelerations of 2.50 $m/sec^2$. The Tokyo (1923) and Avezzano (1915) earthquakes probably gave rise to accelerations reaching up to 4.50 $m/sec^2$.

The duration of an earthquake is short. At Mino Owari in the Japan the total duration of the shocks registered in the 1891 earthquake was of the order of 28 seconds; the period of duration may have reached 30 seconds in the Irpino earthquake of 1930. But other disastrous earthquakes had much shorter durations—that of Tokyo (1923) lasted for 4 to 5 seconds and that of Avezzano (1915) for 6 seconds.

The period of vibration of terrific earthquakes ranges between 0.4 and 1.8 seconds.

Starting from the data mentioned above different investigators (and in particular some of those who have studied the action of seisms on dams) have attempted to find out if the vibration due to a given

*G. B. ALFANO: Modern Seismology, Milano, Hoepli, 1910.
C. S. FOX: Engineering Geology 1935, Cap. VIII p. 168. e F. Penta, op. cit.

earthquake was in resonance with the natural period of vibration of a given strucure.

But such an investigation seems irrelevent. Firstly the vibration due to an earthquake is irregular (as already shown before, *vide* Fig. 1); secondly resonance is only caused by vibrations which are repeated many times and not those that last for very short periods only.

5. From the geological-technical point of view, the principal ideas with which we are concerned are the following:

If the zone in which a construction is to be put up is within the epicentral area of an earthquake of degree higher than X, every precaution is insufficient.

The phenomena observed in such an area are awe-striking (such as modifications in the configuration of the ground, sliding of mountains, opening of chasms). So it is not possible to design a building capable of resisting such violent actions. On the other hand there is only a small probability that a given area, which is generally to be called seismic, should coincide with the epicentral area and that the latter should possess degree of intensity higher than X. Therefore, excluding this exceptional case, a knowledge of the geological and morphological characteristics of an area facilitates much a forecast of the effects of a possible earthquake shock.

In fact, outside the epicentral area, an earthquake does not generally show a disastrous effect, but that effect may increase due to the geo-morphological characteristics of the layers traversed.

The intensification of a shock, caused by the character of the local superficial rock formations, is negligible for the lapideous rocks, is one to three degrees of the Mercalli scale for soil consisting of the disaggregation of lapideous rocks, and of one to four degrees of the Mercalli scale for ground consisting of non-cohering loose material.

This influence of the nature of ground and of the regional and local geological and morphological characteristics is schematically illustrated in Figs. 2, 3, 4, 5 which are reproduced here. In general the strongest effects are seen on narrow and isolated peaks, on the edges of terraces, on steepslopes on declivities consisting of narrow argillaceous or alluvial borders superposed on lapideous formations, or, still more, on formations which are liable to get cracks.

But even in regions which are horizontally level, but where the subsoil consists of non-cohering loose materials (especially if much compressible) and having small thicknesses, a seismic shock, acting as a vibrator, may lead indirectly to catastrophic effects on constructions of which the foundations have not been suitably laid.

The danger of serious damages is still greater when the compressibility is notably non-uniform along different directions in the horizontal plane.

As regards the question of the lines of propagation of seismic waves, great surfaces of discontinuity in the subsoil (faults, fractures, stratigraphic non-uniformity) generally increase sensibly the seismic effects; folds attenuate the effects to a higher or lower degree (by damping), according as the direction of propagation is perpendicular or parallel to the axis of the fold.

Regions which attain a state of tension due to a seismic effect may become secondary centres of an earthquake, etc.

All the above considerations are sufficient to explain why the pattern of the iso-seisms over a region affected by an earthquake is generally so complicated.

6. The preceding remarks apply to all (open air) constructions in general and for dams in particular.

As regards the latter, we do not yet possess sufficient analytical studies from the view-point of seismic effects.

C. Guidi in 1930* made the following remark in regard to the stresses caused in dams by seismic waves—"The fact that in Italy the Corfino and Scoltenna Dams behaved well at the time of the Garfagnana earthquake is no proof that other dams would have behaved just as well; in reality, the small mass of the two dams and their metallic armature assisted much towards the mintenance of their stability during the earthquake". The same author, after summarising the investigations made and the conclusions reached for designing suitably the dimensions of the Pasadena Dam in California, made the following suggestion about taking account of the seismic stresses. "In the regions subject to earthquakes, when designing barrage dams for the formation of artificial lakes, it is necessary to keep in mind that the external vertical forces may be increased by 50 per cent due to an upheaval earthquake, and that horizontal thrusts may arise equivalent to ⅛ of the weight of the masses caught by an undulatory earthquake".

A. Marin, a collaborator of M. Lugeon, studied 23 cases of collapsed dams and found that 19 collapses were due to the slipping mechanical characteristics of the foundation rock, and so due to geological causes. Only four collapses were due to an error of calculation or of construction**. There was not a single case of collapse by seismic action.

R. F. Legget (1939)† attaches great importance to the problem of seismic effects in the designing and dimensioning of dams, and is of the opinion that in earthquake regions seismic stresses should be taken into account even if the foundation beds consist of very solid lapideous rocks.

He records that generally the earthquake factor is considered as equivalent to a horizontal acceleration equal to 1/10 of the gravity, and that the vertical component may be neglected.

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*GUIDI, C. : Should seismic stresses be taken into account in the construction of dams? L'Ingegnere, giugno 1930, p. 375.

**LUGEON, M. : Barrages and Geology. Librairie de l'Universite, Losanna 1933.

†Geology and Engineering. McGraw-Hill, New York & London, 1939 (p. 327-333)

He draws attention to the investigations carried out by H. M. Westergaard for determining the natural period of vibration (about 0.16 sec.) of the Morris Dam (in S. Gabriel Canyon, near Azusa in California, height about 100 m) resulting from the hydrodynamic (seismic) effect of the vibrations of the water of the respective reservoir.

He refers also to the studies made in connection with rock-filled dams in Chile, constructed in earthquake zones.

Legget seems surprised that, as so many old dams were destroyed by seismic action, the dams more recently constructed in a direction perpendicular to faults have resisted so well to earthquake shocks.

He mentions in this connection the earth-filled dams constructed (between 1870 and 1876) over the Sant' Andrea fault in California over an argillaceus formation. During the San Francisco earthquake (degree X of the Rossi-Forel scale) which produced movements along the fault, the two dams did not collapse but were only found deformed with a relative displacement of over 3 metres. A gravity dam built in concrete (in 1877 and enlarged in 1888-99, the Crystal Spring Dam) located at a distance of only quarter mile from the same fault, did not suffer any injury.

Legget then refers to the means (especially the insertion of joints, suitably inclined and bituminized) which should be employed to bind the dams in order to prevent as much as possible the probable relative displacement of the foundation rock and of the imposts under the action of seismic forces.

G. D. Louderback*, in his general description (1950) of faults and their effects on the ground and on engineering works near to faults, distinguishes "active" faults from those for which it can be established that relative movements took place only during the earliest geological periods. With special reference to the California fault, he indicates the movements which faults may undergo and notes that "active" faults are, in general, the origin of seismic waves.

In general, therefore, one should avoid to locate engineering works in fault zones which are admitted as "active". When this is not possible, it is necessary to adopt measures which will ensure the maximum security factor. Louderback quotes some examples of reservoir dams supported on imposts over fault zones.

To mention a special case, the Temescal Dam was situated in a fault zone which suffered displacements on the occasion of the San Francisco earthquake (1906). But the dam itself which was an earth-filled structure did not suffer any injury.

In seismic zones, or when situated over active faults, it is advisable to adopt deformable barrage structures, since even in case of large deformations, ruptures will only take place gradually.

*LOUDERBACK, G.D.: Faults and engineering geology in "The Geological Society of America", Application of Geology of Engineering Practice, 1950.

It is also advisable to adopt the measures necessary for the protection of the impermeable nucleus of the structure from the action of waves that are likely to be formed in the reservoir. The impermeable nucleus should have considerable dimensions and the declivity of the faces of the dam should be very slight.

E. B. Jr. Burwell and B. C. Moneymaker* in their treatment of the subject of geology in dam construction (1950) make no mention of the seismic effects on dams.

F. D. Kirn† suggests that account should be taken of the overload due to direct seismic action (acceleration) on the structure, as well as of that due to the indirect action transmitted by the mass of water (or mud) pressing on the dam. He also draws attention to the need of keeping in mind the effects of resonance.

J. R. Schultz and A. B. Cleaves (1955), in the chapter on dams found in their Tex Book of Geology for Engineering‡, treat the subject of "seismic forces". They begin by remarking that few problems are so difficult to handle as that of the possible effects of seismic forces on dams. Records show that well-designed dams have very seldom failed to resist even violent earthquake shocks. The Crystal Springs Dam is located at a distance of about 100 metres only from the fault which was "responsible" for the 1906 San Francisco earthquake, but the structure survived the shock without any injury. This dam is constructed in concrete and belongs to the type of gravity arch dams. For the deformation of the upper Crystal Spring Dam (earth-filled and about 25 metres high), *vide* the paper by Legget (§) previously referred to.

During the Santa Barbara (California) earthquake, the Gibralter Dam (an arch type structure) suffered such a heavy shock that the superintendent who was accidentally on the top of the dam, found difficulty in keeping his footing. Nevertheless the dam did not suffer any injury.

On the other hand, there is no doubt that earthquakes reduce considerably the margin of security and it is therefore necessary to make a careful examination of the risks involved and of the precautions which it may be possible to adopt for increasing the security.

7. According to Schultz and Cleaves, the accelerations to which earthquakes give rise depend on the intensity of the earthquake and on the sum total of the "local factors".

Accelerations which reach very nearly the value of gravity probably arise in the case of sufficiently violent shocks but their effect is

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*BURWELL, E.B. Jr. & MONEYMAKER, B.C.: Geology in dam construction, in "The Geological Society of America". Application of geology to engineering practice, 1950.

†"Criteria for the design of gravity and arch-type dams" published by "Bureau of Reclamation" (Denver, Colorado, October 1953).

‡Geology in Engineering. Ed. J. Wiley & Sons & Chapman & Hall, New York & London, 1955.

§Cited above.

very much localised. For "solid" rocks lying at a distance of a few miles from the epicentre, it seems probable that the accelerations are seldom greater than 0.1 of the value of gravity. In the case of loose material, the value may be somewhat greater. In the U.S.A. the common practice is to adopt a security factor of about 0.1 of the gravity for horizontal forces. Vertical accelerations are considered to be negligible. If a dam is to be situated near an active fault, or its foundations laid on loose material, higher security factors are probably necessary. Dams should not be constructed in such localities if it is probable that the consequences of the deformations will be serious. It is also advisable to avoid localities where there are faults. But unless one knows for certain that it is a question of an active fault, such localities may still be treated as admissible, provided suitable treatment of the foundations is adopted and selection made of suitable structural types.

The Pine Canyon Dam near Pasadena in California is placed over a fault which is considered to be inactive. Nevertheless, as an additional security factor, a joint has been constructed in the masonry above the fault so that, in case of a dislocation of the fault, the flexibility of the joint might serve to prevent or minimise the occurrence of fissures in the concrete.

The Rodriguez Dam in Mexico is located over a fault which passes across the bed of a stream. In this case a special type of arch was constructed at the lowest point of the surface downstream to serve as an additional security factor. The purpose of the arch is to make uniform the part of the load supported by the less resistant rock of the river-bed and transfer the rest to the sound rock of the sides of the river valley.

We may also mention that M. Gignoux and R. Barbier in their recent book* do not treat of the influence of seismic phenomenon on the stability of dams.

It is not necessary to proceed straight away at this stage to an examination of the department of dams in Italian territories which have been struck by earthquakes of a certain degree of intensity.

It seems enough to refer to the few cases already reported by C. Guidi (1913) for the seismic zone of Garfagnana† and draw

*GIGNOUX, M. & BARBIER, R.: "Geology of Barrages and other hydraulic Works". Ed. Massone, C.; Paris, 1955.

†The Garfagnana earthquake of 7-9-1920 was defined as Catastrophic (degree IX-X). In the region most affected by this earthquake there were two dams namely;

(1) The Corfino Dam of simple arch-type structure 42 metres high max. thickness 2 metres at the summit and 7 metres at the base. It was supported on imposts in a massive (exotic) block of greenish rock lying within the scaly argillacious formation. This dam did not suffer any deformation though not far away, the village of Villa Collemandina was completely destroyed.

(2) The Scoltena Dam across Rio Lunato, a multiple arch-type structure, with masonry spurs, about 20 metres high implanted on sandy foundations, notwithstanding this the dam did not suffer any injuries, though it fell within the epicentral area. The latter dam is built in reinforced concrete, while from the description of the former as given by its designer Engineer Omodeo in "L" Industria". January 1918 (p. 18) it appears that the Corfino dam is in plaster (2.5 cm thick), reinforced with streched steel sheets.

attention to the damages suffered by the dams of Alto Fronto (Scandarello) and of Vomano (Campotosto, Provvidenza), and their respective works, on the occasion of the Gran Sasso earthquake of September 1950. It should be noted that all the works were situated in an area which was then classified as epicentral of degree VIII*. It is also necessary to add that both the foundations and the imposts of the dams lie in the same arenaceous formation† which is considered, as regards seismic effects, as a lapideous formation.

The Turano and Slato dams which fall in grade VI zone of the same earthquake, and which were also supported on imposts on lapideous formations, also suffered no derangements or disturbances.

What we have written so far treats of generalities.

8. Let us now enter into some particular aspects of the seismic influences on dams. Some investigators are of the view that a seismic shock should not affect dams in the direction normal to the face, that in the longitudinal direction; that is, in the passage from the structure to the ground; and caused by the sudden variation in the elastic characteristics between the structure and the ground. In view of this, one would be led to employ, depending on the foundation rock, structures with elastic characteristics changing gradually from those of the ground to those of the main body of the structure

In the case of valleys caused by fractures, the orientation with respect to the fracture and to the direction of propagation of the seismic waves should be carefully examined, taking into account the influence of such fractures on the intensity of seismic effects, of which we have already treated above, and shown graphically in figures 2 to 5.

In this connection, we may refer to the measures adopted by American constructors of certain dams in cases similar to those above.

As regards the danger of the sinking or sliding of the banks under the action of earthquakes in formations of a small degree of coherence, it should be noted that a dam constructed as a barrage serves also the purpose of a support to the banks themselves.

But this case of easily breakable banks is not quite relevant to our argument which we are treating. For we are only considering masonry dams for which the fundamental assumption is that the banks as well as the section to be covered by the barrage consist of lapideous formations having a high degree of cohesion, and therefore not liable to slip.

*V.D. Di Filippo & L. Marcelli: A study of the earthquake of Gran Sasso (Italy) of 5th September 1950.

†This region forms a part of the Miocene Marl-sand (Pontic Molassa) which surrounds on the north and east the massif of Gran Sasso. This formation consists of layers of sandy rock of different thicknesses (few decimetres to a few metres), sometimes alternating with banks of clay marl and of bluish clays" (vide the recent investigation of J. Demangeot M. Manfredini, etc).
The formation has a thickness of over 1000 m and is everywhere more or less incongruous with the underlying strata.

The Compostosto Lake is a depression filled with quarternary deposits.

Suddenly to the east of Compostosto is found a fault directed approximately N.S.

In the case of sections consisting of loose formations which are compressible (uniformly or non-uniformly), the seismic problem takes another form, especially because, in the case of these types of ground, we can only think in terms of deformable structures.

In this connection R. W. Clough and Z. Pirtz* have recently described the experiments that they carried out on models of rock-filled dams in order to measure the resistance of this type of structure to seismic action.

First of all, they determined the conditions which a model should satisfy in order that the dynamical similarity between the model and the dam should be accurately reproduced. The model was then subjected to dynamic forces induced by blows of a beating mass which struck an oscillating table, on which the model was placed.

They tested two types of structure (one with inclined impermeable nucleus and another with vertical impermeable nucleus) under different conditions of filling of the reservoir upstream. The models were subjected to accelerations lying between the values 0.08g and 1.23g ($g =$ acceleration of gravity).

No sinkings or collapses sufficient to load to the total destruction of the structures was noted during the testing, but only deformations of the faces and sinkings of the crest of the dam.

From these analyses of the displacements suffered by the model dam it came out that there was a certain difference of phase between the movement of the base of the dam and its crest.

Permanent deformations of the structure only took place for accelerations greater than 0.4g.

According to the authors the high resistance to seismic effects shown by rock-filled dams seems to be due to the fact that loose materials show a higher resistance to dynamical forces than statical. Further, dams of this type are in a condition to undergo large deformations and to sustain considerable yieldings without cracking up.

In the opinion of the authors the usual methods of calculation for taking account of dynamical actions (method based on considering a statical force equal to 0.1gM where M is the mass of the dam) has no value from the practical point of view. This is because such a method of calculation does not take account of the variation in direction of the accelerations of an earthquake.

No difference of department between the models with inclined impermeable nucleus and those with vertical nucleus were found by the investigators.

J. W. Spielman, in discussing the communication of the two authors just mentioned, draws attention to the fact that in order to render similar all the conditions between the models and the dams, it is necessary to reduce the breaking load of the materials employed for

*CLOUGH. R.W & PIRTZ. Z.: Earthquake resistance of rock-fill dam Proc. ASCE, SM 2, p. 941, 1956.

the construction of the models. In fact, in the prototypes a good number of the yieldings which are noted are due to the breakage of the rocks of which the dam consists, at the points where the rocks are in contact among themselves. These conditions are not realised in the models.

N. N. Ambraseis, who also took part in the discussion on the paper by Clough and Pirtz, did not agree as regards the reasons for the high resistance of rock-filled dams. In particular he did not admit that loose materials could present a higher resistance to dynamic actions, except under very special conditions. According to Ambraseis, the high resistance to seismic actions which was under discussion is solely due to the possibility of rock-filled dams bearing large deformations without breakage, especially if the foundation rock is sufficiently solid. The essential factor which is responsible for the resistance of the structure is the foundation rock. If the foundation rock is somewhat soft it is to be feared that deformation of the structure will take place as an effect of excessive yieldings and the creation of discontinuity in the foundation rock itself; the latter will lead to the sinking of the faces of the dam due to decrease of the supporting power of the ground constituting the foundation.

Different criteria than those mentioned seem therefore to hold true for dams consisting of deformable structures.

In these cases, however, especially when dealing with earth-filled dams, the height of the flood bank above water line should be proportioned taking into consideration the seismic disturbance of the free surface of the water. In this connection, and by way of an example, M. Baratta (1936, p. 28), referring to sea-quakes, observed that "in the Argenta earthquake of 1624 the waters of Po-di-Primaro and the neighbouring river courses, as though lashed by a fierce tempest, rose into huge waves which beat menacingly on the high banks".

We may thus conclude that the main criterion to be followed in selecting the placement for the construction of a dam in a seismic region is reduced to the searching for a locality where the conditions which increase the intensity of the seismic effect are, as far as possible, absent.

The general geologisal characteristics which are the most unfavourable from this point of view have already been explained and illustrated in figures 2—5. Recalling that, as already mentioned, we are only treating masonry dams, which presuppose lapideous foundation rocks, we need not consider the unfavourable seismic effects which relate to loose material formations.

But even assuming that we are only dealing with lapideous formations, it is still advisable to avoid the placement of masonry structures in correspondence to surfaces of discontinuity along which complications (like refraction, reflection, etc.) may arise in the ordinary and simple process of propagation of a train of elastic waves. The effect of such complications will be felt on the surface of the ground and on the works that it supports in the form of a series of impulses and vibrations.

Surfaces of discontinuity are not to be limited to fractures and active faults only, but they may consist of contacts of any sort between rock formations of markedly different elastic characteristics.

A preliminary survey, carried out by using the methods offered by modern geo-seismics, followed by a suitable interpretation of the results, should give the designer valuable elements for judgement.

It should be well understood that all that we have been saying is valid for areas outside those which are epicentral and of degree higher than X. We do not have any pretention of giving criteria for guaranteeing the stability of a dam (or other similar construction) erected in a place where the original shock (not augmented by the nature of the ground) deforms, breaks, or dislocates straightaway even lapideous formations. What we mean is, that the case of the placement of a dam in an epicentral zone of degree higher than X is altogether excluded.

For obtaining a higher degree of safety in cases where the lapideous formation has favourable configuration and arrangement, but an original shock may be felt which is more than sensitive though not catastrophic, it is necessary to take account of the stresses likely to arise in the manner given below.

9. We have so far recalled the various investigations on the seismic action on dams, carried out on the basis of an analysis of the different deportments of rock formations and that of the works already constructed. Let us now determine if it is possible to ensure the safety of a dam by prescribing special safety factors for calculating the proportions of the dam. It is obviously to be borne in mind that the safety factors only refer to earthquake shocks outside the epicentral zone (at least for earthquakes of degree higher than X).

Before taking up the problem we wish to refer to the restrictive specifications which are in force for antiseismic buildings, as well as to the suggestions of the different authors who have treated the seismic stresses in dams from the point of view of engineering calculations.

The rules for antiseismic buildings suggested by the California (1906) and the Calabrian-Sicilian (1908) earthquakes are based on the evaluation of the accelerations caused by the different shocks. The accelerations are less than 1 m/sec.$^2$ for earthquakes up to degree IX inclusive. But they may attain values of 2.5 m/sec.$^2$ for earthquakes of degree X and 5 m/sec.$^2$ for earthquakes of degree XI.

In Italy the "technical standards for buildings with special specifications for localities subject to earthquakes" at present in force (Legislative Decree No. 2105 of 22-11-1937) lay down the following (article 31) : In order to take account of the seismic acceleration due to undulatory earthquakes, the total load is to be taken as equal to the sum of the actual load plus 1/3 of the accidental overload increased by 40 per cent in those cases where the total obtained is not less than the sum of the actual load and the accidental load.

The following forces are also to be considered : Horizontal forces applied to the masses of the different parts of the building, arising from the seismic accelerations transmitted to the building by the undulatory earthquake. These forces should be considered as acting in both senses, whether in the longitudinal or the transversal direction. The ratio between the horizontal forces and the weights corresponding to the masses on which the forces act should be taken as equal to 0.10, whatever may be the height of the building and the number of storeys. For the calculation of the horizontal forces the accidental load should be limited to 1/3 of the maximum value assumed for the load of the individual structures.

The above prescriptions are valid for seismic localities of category I. For those of category II we are asked to assume for the vertical forces an overload of 25 per cent (instead of 40 per cent), and for the horizontal forces a ratio of 0.05 (instead of 0.10).

It will be seen that the present-day legislation relating to buildings is much more severe for the upheaval than for the undulatory earthquakes : the load taken into account for upheaval shocks (actual weight plus 1/3 of the accidental load) is essentially the total effective load (because if any ceiling is calculated at 250 kg/m$^2$ it does not exactly follow that the load acting on the ceiling is always 250 kg/m$^2$, but that, on the average, the load acting on all the ceilings of the building will have a much smaller value); now the total effective load is increased by 40 per cent and so a vertical acceleration of 4.00 m/sec.$^2$ is provided for. On the other hand the horizontal acceleration provided for is only 1.00 m/sec$^2$.

This difference is partly due to the fact that upheaval shocks generally have a more serious effect on buildings than undulatory shocks. Further the prescribed instruction of taking into account 1/3 of the accidental load is, for undulatory shocks, equivalent to increasing conventionally the accelerations. (This is because ordinarily the accidental load is not restricted to the walls but by horizontal vibrations may pass to the floor, while for upheaval earthquakes the accidental load acts directly as an overload).

We should observe that these standards for antiseismic buildings have been modified several times and the specifications now prescribed are less severe than those of the older standards.

In fact the Legislative Decree No. 2089 of 23-10-1924, now replacd by the 1937 Decree, prescribed the taking into account of the actual weight plus the whole of the accidental weight (article 28), and marked an overload of 50 per cent for upheaval earthquakes and of 12 per cent (about ⅛) for undulatory earthquakes, in the case of buildings not higher than 12 m; for all other buildings it prescribed, in the case of undulatory earthquakes, an overload of 18 per cent (about 1/6). The same specifications were also given in the temporary Legislative Decree of 19-8-1917, article 229.

The tendency to prescribe less rigorous specifications may be justified by the fact that no precaution is posible in epicentral zones of earthquakes of degree higher than X and, therefore, it is sufficient to limit the prescriptions to shocks which cause accelerations of less than 2.5 m/sec$^2$. The inconsistency of providing unnecessarily high accelerations for upheaval earthquakes still remains in the current standards.

10. Several investigators have taken up the problem of including the effect of seismic actions in the calculations relating to dams.

In Italy the problem of calculations relating to gravity dams was taken up by Puppini in a paper dated 1921*. In this paper no account is taken of the seismic action of the reservoir water against the dam (a point which was studied only 12 years later by Westergaard). Essentially, two points result from Puppini's paper:

(1) When it is desired to construct a gravity dam capable of resisting seismic actions, both when the reservoir is full and when empty, it is necessary to give up the usual form of the dam, that is the form with a vertical face upstream.

(2) A dam with the two faces equally inclined is best adapted to resist seismic action (producing accelerations up to 2.50 m/sec.$^2$) if the triangular profile selected has the base equal to the height of the triangle.

Guidi† treated the problem in a very short note in 1913, he also ignored the seismic action of the water against the dam. As already mentioned, Guidi in this article, though mentioning that the Corfino and Scoltenna Dams showed a good deportment in the Garfagnana earthquake, suggested that in the calculations for dams (of any type) we should observe the current standards for buildings (and therefore referred to the Legislative Decree of 1924 mentioned in the preceding section, but he limited the thrusts for undulatory earthquakes to $\frac{1}{8}$ of the weight).

As far as we are aware, no writer has yet remarked that, while for gravity dams both undulatory and upheaval shocks cause anxiety, arch type dams (Corfino type dams) are subject to grave risks only in the case of undulatory shocks.

In America prescriptions relative to the seismic effects on dams have been included in several treatises.

Davis ‡, referring to gravity dams, observes that two types of horizontal seismic forces are to be considered—those due to the inertia of the dam and those due to the pressure of the water. For the latter, we may adopt for the foundation plane, according to Karman§, $F=0.555\alpha\gamma ah^2$ ($\alpha$ being the ratio between the seismic acceleration and the value of gravity, $\gamma a$ the specific gravity of

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*U. PUPPINI : "Massive dams in Seismic Zones." "Memorie della R. Accademia delle Scienze dell'istituto di Bologna", Serie VII, Tomo VIII (1920-1921).

†C. GUIDI: loc. al n. 6.

‡Handbook of Applied Hydraulics, C. V. Davis, editor-in-chief, McGraw-Hill book company, New York, 1942, page 137.

§The formula was suggested by Karman in the discussion following the reading of his paper by Westergaard.

water and 'h' the height of the dam). The force F is applied at a distance of $4h/3\pi$ on the foundation plane. Davis considers, however, that besides these horizontal forces we should also take account of vertical forces (relating to undulatory shocks).

In the treatise by Creager, Justin and Hinds* it is stated that in regions where earthquakes are possible or probable, the calculations should take into account the seismic stresses even if it has been observed that some dams did not suffer injury during recorded earthquakes. In this connection he mentions two dams—the arch gravity dam of Crystal Springs which resisted well the California earthquake of 1906 and the arch type Gibralter Dam, which resisted well the Santa-Barbara earthquakes.

After these preliminary remarks he suggests that seismic stresses should be treated in the usual way (both as regards the action of the water and as regards the acceleration of the material). Following Dewell† he suggests that the earthquake factor to be adopted (in the USA) should be an acceleration of about 1 m/sec$^2$.

The Bureau of Reclamation published a monograph in October 1953 (written by F. D. Kirn) giving criteria for design of gravity type and arch type dams.

In this monograph it is stated that the maximum acceleration to be considered is about 0.3 g, and it is suggested that account of seismic shocks should be taken in the following manner:—

(A) *Undulatory shocks* :

(1) For dams which have their upstream face vertical or inclined the variation of the hydro-static pressure due to an earthquake is given, in terms of the depth of the reservoir, by the equation as given below:

$$P_e = C \propto \gamma_a h$$

$$C = \frac{C_m}{2}\left[\frac{\gamma}{h}\left(2 - \frac{\gamma}{h}\right) + \sqrt{\frac{\gamma}{h}\left(2 - \frac{\gamma}{h}\right)}\right]$$

where the symbols have the following significance:—

$P_e$=The normal pressure on the face;

$\propto$=The intensity of the earthquake=acceleration of the earthquake/acceleration of gravity;

$\gamma_a$=Specific gravity of water;

h=The maximum depth of the reservoir;

$\gamma$=The vertical distance of the surface of the reservoir from the level under consideration; and

$C_m$=Non-dimensional coefficient, the value of which can be taken from the graph, Fig. 6.

The formula was obtained on the analogy of a similar equation in electrical theory, adopting the simplifying hypothesis that water is non-compressible.

*Creager, Justin and Hinds: Engineering for Dams, Vol. II, New York, 1945, pp.279-286.

†Dewell, H. D.: Earthquake-resistant Construction, nel' "Engineering New-Record", April 26, 1928, p. 650.

(2) If the dam has its upstream face at an inclination varying with height and the vertical portion extends over more than half the height, the whole face may be considered as vertical. If on the other hand the vertical part is less than half the height, a mean inclination should be obtained by joining the highest upper pond level with the upstream base of the dam.

(B) *Upheaval shocks* :

(1) The component of the pressure of water normal to the face should be altered by applying the appropriate acceleration factor.

(2) The unit weight of the concrete should also be modified by applying the same factor*.

In Germany Kelen† refers to the action of earthquakes and suggests the use of the formula due to Karman and Westergaard, of which we have already spoken. He gives an example of the calculation for an undulatory shock.

Toelke‡ gives only a general treatment of the problem, remarking that arch type dams show an excellent deportment during earthquakes, that gravity dams behave less well, and rock-filled and dry masonry dams still worse. He thinks that earth-filled dams would be the best type.

Press§ remarks that seismic effects should be taken into account only in the zones subject to earthquakes. He suggests the adoption of an acceleration of about 1 m/sec.$^2$ for the body of the dam; the increase of pressure due to the inertia of the water should be calculated, he suggests, with the formula $p=0.08\gamma a\sqrt{(2H-y)y}$, where H is the height of the dam and y the vertical distance from the summit of the point considered. He holds that the stresses calculated with the overload indicated by him may be greater than those generally admitted as possible by about 25 per cent.

The German Regulations (DIN 19,700, February 1953) do not make any mention of earthquakes. In France, Bourgin's treatise¶ on calculation for dams takes account of the seismic acceleration on the material of the dam and even works out a numerical example in this respect, but he only makes mention of the inertia of the water, with the remark that the problem is too difficult for theoretical treatment.

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*The design of some of the modern American dams takes account of all types of the additional stresses arising from earthquake shocks. Thus the Morris dam of California, which is of the gravity type and 97m high has dimensions which are about 15 per cent larger than those which it would have in the absence of seismic actions. cfr. S. B. Morris & C. E. Pearce: A concrete gravity dam for a faulted mountainous area. Eng. New-Record, Vol. 113 (1934), pp. 823-827.

†N. Kelen : "Gravity type barrage masonry dams and massive weirs." Berlino, Springer, 1938, p. 17 & p. 36.

‡A Ludin : "Constructions for water power stations", 2nd half, 1st part "walls across valleys, barrage dams and barrage walls." Friedrich Toelke, Berlino, Springer, 1938, pp. 110-113.

§H. Press: Barrage dams and water power works, Part 1, Talsperren Berlino, W. Ernst, 1953, pp. 16 & 18-19.

¶A. Bourgin: A course of calculations for barrages. Parigi, Eyrolles, 1948, pp. 29-31.

11. It will be observed that in all the treatises cited in the preceding section, the only theory mentioned is that of Westergaard Since this theory has not been included in any Italian publication, we think it is appropriate to give a brief exposition of it in this paper.

The problem to be solved may be posed as follows*.

Let us consider a dam with vertical face upstream. The dam, which is solidly attached to the ground, commences to vibrate at the same instant as the latter, while the water remains still, due to inertia. On this account the dam behaves towards the water mass as the piston of a pump and causes a motion of the liquid which is in contact with the upstream face of the dam. This is the fundamental cause of the excess of pressure exerted by the mass of water.

Westergaard takes up the solution of the problem on the analogy of the method employed in hydrodynamics for the study of expansion waves†.

Using the notation ordinarily employed in Italy we have

$$\frac{\delta\rho}{\delta x} = e \, \frac{\delta^2 \varepsilon}{\delta t^2}, \qquad \frac{\delta\rho}{\delta\gamma} = e \, \frac{\delta^2 \eta}{\delta t^2} \tag{1}$$



where, since the displacements are small, we have written $\frac{\delta\xi}{\delta t}$ instead of $\frac{D\xi}{Dt} = u$; $\frac{\delta\eta}{\delta t}$ instead of $\frac{Dn}{Dt} = v$; and similarly $\frac{\delta u}{\delta t}$ instead of $\frac{Du}{Dt}$ etc.

We, therefore, have $p = \varepsilon\theta$, where $\varepsilon$ is the modulus of compressibility and

$$\theta = \frac{\delta\xi}{\delta x} + \frac{\delta\eta}{\delta\gamma} \tag{1'}$$

so that:

$$\rho = \varepsilon \left( \frac{\delta\varepsilon}{\delta\gamma} + \frac{\delta\eta}{\delta\gamma} \right) \tag{2}$$

The motion is governed by equations (1) and (2). The value of $\dot{\varepsilon}$ for water is known to be equal to $2.07 \times 10^8$ kgm$^{-2}$.

The limiting conditions are:—

$$\rho = 0 \quad \text{for } \gamma = 0$$

$$\eta = 0 \quad \text{for } \gamma = h \tag{2'}$$

$$\xi = \frac{\alpha g T^2}{4\pi^2} \cos \frac{2\pi t}{T} \qquad \text{for } x = 0 \tag{2'}$$

where $\alpha g$ is the (horizontal) acceleration due to the earthquake ($\alpha$ being a pure number $< 1$), t the time, and T the period.

*H.M. Westergaard : Water pressure on dams during Earthquakes. "Trans. ASCE", 1933, p. 418. *Vide* also the "Discussion" which followed the publication of this article, and particularly the letters by K. V. Karman, Bauman, etc.

†V. per es H. Lamb: Hydrodinamics, Chapter X.

It will be noted that the first limiting condition leads to a further simplification (analogous to that introduced by Poisson in the theory of vibration waves), namely to put p=0 for y=0, while p=0 for y=η.

If, besides the two preceding limiting conditions, we make the further assumption that when x tends to infinity, p tends to zero, Westergaard is able to put ε and η in the following form, which satisfies all the conditions imposed:—

$$\xi = -\frac{\alpha g T^2}{\pi^3} \operatorname{Cos} \frac{2\pi t}{T} \sum_{n=0}^{\infty} \frac{e^{-q_{2n+1}}}{2n+1} \operatorname{Sin} \frac{(2n+1)\pi y}{2h}$$

$$\eta = \frac{\alpha g T^2}{\pi^3} \operatorname{Cos} \frac{2\pi t}{T} \sum_{n=0}^{\infty} \frac{e^{-q_{2n+1}}}{(2n+1)C_{2n+1}} \operatorname{Cos} \frac{(2n+1)\pi y}{2h} \qquad [2']$$

where :

$$C^2_{2n+1} = 1 - \frac{16\,\varrho\,h}{(2n+1)^2 \varepsilon T^2}$$

$$q_{2n+1} = \frac{(2n+1)\pi C_{2n+1} . x}{2h} \qquad [2'']$$

It follows from equation (2) above that

$$\rho = \frac{8\alpha\gamma h}{\pi^2} \operatorname{Cos} \frac{2\pi t}{T}$$

$$\sum_{n=0}^{\infty} \frac{e^{-q_{2n+1}}}{(2n+1)^2 C_{2n+1}} \operatorname{Sn} \frac{(2n+1)\pi y}{2h} \qquad [3]$$

where γ is the specific gravity of the water.

Let us consider the pressures exerted on the dam. These are obtained from equation (3) and have maximum values for t=0, T, 2 T, etc. The pressure is maximum at the bottom and in correspondence with the dam, where we obtain :

$$p_o = \frac{8\alpha\gamma h}{\pi^2} \sum_{n=0}^{\infty} \frac{(-1)^n}{(2n+1)^2 C_{2n+1}} \qquad [3']$$

it is nil at the surface and it varies along the face very nearly in accordance with a parabolic curve. In fact, Westergaard suggests an approximate formula, which employing the metre and the second as units may be written as follows :—

$$p = \frac{0.90\,\alpha\sqrt{hy}}{\sqrt{1-7.74\left(\frac{h}{1000\,T}\right)^2}} \; tonne/m^2 \qquad [3'']$$

tonne=(metric) ton.

The trouble with the above formula is that it is dimensionally non-homogeneous: we have to express h in m, y in m and T in sec.

It seems, therefore, simpler to establish an empirical formula based on the two following points :

(1) that the variation of p can be approximately represented by a parabola with vertex at the surface of the water and its axis vertical; and

(2) that consequently it is sufficient to determine the value of p at the bottom to obtain all the parameters of parabola.

Now equation (3′), if we take account of the values of the coefficients of the series, we obtain :—

$$p_o = \frac{8\alpha\gamma h}{\pi^2} \cdot \frac{1}{C_o} \quad [4]$$

where

$$C^2_o = 1 - \frac{16\,\varrho h^2}{\varepsilon T^2} \quad [5]$$

Since T=1 sec, $\varrho$ =100, $\varepsilon=2.07 \times 10^8$, it follows that $C^2_o=1-0.00000773\ h^2$ and therefore, for heights of water less than 150 m, $C^2_o$ is practically equal to 1. For h=150 m, $C_o^2=0.83$; $C_o=0.91$. If h=200 m., $C_o^2=0.70$ and $C_o=0.84$.

If we put $C_o=1$ and $\alpha=0.1$ (acceleration of 1 m/sec²), we finally obtain :—

$$p_o \approx 0{\cdot}08\gamma^{\alpha}h \quad [6]$$

If on the other hand $C_o=0.8$, we obtain :

$$p_o \approx 0{\cdot}1\gamma^{\alpha}h \quad [7]$$

Since p varies very nearly in accordance with a parabolic law, we may write for depth y :—

$$p = 0{\cdot}1\gamma^{\alpha}\sqrt{hy}; \quad [8]$$

the above expression while representing a rough approximation gives an extremely simple formula.

The total pressure will be given by :—

$$p = \int p\,dy = 0{\cdot}067\ \gamma^{\alpha}h^2 \quad [9]$$

and will be exerted at a height 2h/5 from the base.

If we wish to reduce the calculations to a still more elementary form, we may proceed as follows :—

Let us consider a pressure $p_1$ which is distributed uniformly on the upstream face of the dam and find what value should be given to $p_1$ in order to produce the same effect as p. It is sufficient to observe that the action of p essentially consists in producing an additional moment on the base. This moment may be expressed, assuming the simplifications already introduced, by $\frac{2h}{5}p = 0.0267\gamma a h^3$, while the uniform pressure $p_1$ produces a moment=$\frac{1}{2}$ h $p_1$ h writing the two moments as equal, we obtain :—

$$p_1 = 0{\cdot}053\ \gamma\alpha h^{*}. \quad [10]$$

It follows that the test of stability of the base of a dam in respect of seismic actions caused by the water may be carried out by assuming the following :—That, in addition to the hydrostatic pressure, the upstream face of the dam is subjected to a pressure $p_1$, normal to the face and uniformly distributed on the face. Obviously, the simplification enhances the conditions of stability to be imposed on the horizontal layers of the dam higher than the base.

*This formula differs very little from that given by Karman (for $\alpha = 0{\cdot}1$).

12. Let us now go back to the practical aspect of the problem of the stability of dams when confronted with seismic forces.

We wish to observe in the first place that no interest is attached to the question of designing a dam capable of resisting an earthquake even when the reservoir is dry. In fact a barrage has by itself sufficient stability not to be affected by earthquakes corresponding in intensity to the earlier degrees of the Mercalli scale, we have only to see that the dam is solid enough to resist shocks of degrees IX and X; it is very improbable that any structure will resist the intensities designated as "catastrophe" or "great catastrophe".

Now, if for the intensities within our view, an earthquake causes the collapse of a dam when the reservoir is empty, there is no doubt that the material loss will be heavy, but the consequences of collapse on other works will be nil. If on the other hand the reservoir is full, the collapse of the dam will produce a fearful flood sufficient to augment notably the destruction caused by the earthquake in the zone affected by it. We may recall that the collapse, without any earthquake, of the Gleno and Orba Dams caused very serious destruction with hundreds of deaths. We shall, therefore, only consider the tests for stability against seismic effects in the case when the reservoir is full.

What we need is to establish value of the maximum acceleration to be taken into account in the case of a filled reservoir.

As regards the upheaval wave, we think it is convenient to assume accelerations of an order of magnitude of 2 m/sec$^2$ it may be noted that, when the seismic acceleration acts downwards, our assumption implies that accelerations of about 12 m/sec$^2$ may normally occur. We may, therefore, assume that for very short periods the structure is able to resist accelerations of 12.5 m/sec$^2$ (that is, the dam will be able to resist earthquakes of degree X).

When the acceleration due to the seismic wave is directed upwards, it may be noted that the proposed value of 2 m/sec$^2$ will leave a gravity component equal to about 8 m/sec$^2$ (which value may decrease to about 7 m/sec$^2$ for earthquakes of degree X).

But the stability of the structures is also ensured by other factors; thus in gravity type dams account is always taken of possible falls of the hydraulic pressures below the normal (these constitute an effective safety margin rather than a real water load). For arch type dams, we have already observed that their own weight does not have much influence in producing stresses.

As regards horizontal seismic waves, we think that it is generally sufficient to take into account an acceleration of 1 m/sec$^2$; we have chosen this value not so much because this value has been prescribed in the regulations for buildings, but on account of the following considerations : theory indicates that in the case of an acceleration which is directed downstream, account should be taken simultaneously of the actions of the mass of water and of the mass of the dam, on the assumption that both are in the same phase (as this is necessary when we assume the hypothesis of harmonic vibration); but in an actual case, as the vibrations are very irregular, the two actions will differ in phase. It seems, therefore, that if we take the value of the horizontal component of the acceleration as equal to 2 m/sec$^2$, this would lead to unnecessary high solidity of the structure. On the other hand if a region does not show favourable characteristics, we may provide for an even higher rick (i.e. take into account a horizontal acceleration equal to the vertical, that is, 0.2 g).

These observations, we believe, justify the text of article 25, which is reproduced in the appendix.

13. Let us calculate, for illustration, the changes of thickness produced in the case of adopting the antiseismic rules in a massive gravity type dam with triangular profile.

With the notation employed in figure 7 (and recalling that for an element of a parabola with the vertex at A, length h and ordinate $p_0$, the area equals 2/3 $p_0$h, while the centre of gravity lies at a distance of 2/5 h from the bottom), we have, calculating the moments with respect to 1/3 of the height downstream, the following conditions for equilibrium :—

$$\frac{1}{2}\ \gamma_a h^2 . \frac{h}{3} + \frac{2}{3}\ (0{\cdot}1\gamma_a h^2)\ .\ \frac{2}{5} h + \left(0{\cdot}1\gamma_m \frac{hx}{2}\right)\frac{h}{3}$$

$$= \frac{1}{2}\ \gamma_m hx . \frac{x}{3} \qquad [11]$$

where :

$\frac{1}{2}\gamma_a h^2$ = Thrust of water applied at h/3 from the bottom.

$2/3(0.1\gamma_a h^2)$ = Increase of the pressure of water due to seismic action applied at 2h/3 from the bottom.

$0.1\ \gamma_m \frac{hx}{2}$ = Increase of the pressure in the masonry work due to seismic action applied at h/3 from the bottom.

$\gamma_m \frac{hx}{2}$ = Vertical weight of the masonry applied at h/3 upstream.

It follows from the preceding equation that :—

$$\gamma_a h^3 (1 + 0{\cdot}16) + 0{\cdot}1\gamma_m h^2 x = \gamma_m h x^2 \qquad [12]$$

that is :

$$x^2 - 0{\cdot}1hx - \frac{\gamma_a}{\gamma_m} h^2 .\ 1{\cdot}16 = 0$$

$$x = 0{\cdot}05h + \sqrt{(0{\cdot}05h)^2 + \frac{\gamma_a}{\gamma_m} h^2 .\ 1{\cdot}16}$$

$$x = h\left\{ 1{\cdot}08 \sqrt{\frac{\gamma_a}{\gamma_m}} + 0{\cdot}05 \right\} \qquad [13]$$

Since, in the absence of seismic action, $x = h\ \sqrt{\gamma_a/\gamma_m}$ we deduce that the increase of thickness by undulatory seismic effect will be about 16 per cent.

For upheaval seismic effects, the calculation is even simpler.

Let $\gamma_m$ be the specific gravity of the material. If the acceleration of gravity is reduced by 20 per cent, it means that the specific gravity of the material is reduced to 80 per cent of the original. We shall now find that—

$$x=h\sqrt{\gamma_a/0.8\gamma_m} = 1.118h\sqrt{\gamma_a/\gamma_m}$$

An increase in the gravity would not induce any increase in the dimensions of the base except if the pressure on it attains values higher than the safety load introduced.

14. In order to assess correctly the degree of seismicity of a given locality it is necessary to refer to the official catalogues of recorded earthquakes in the communes of Italy, and to the compilations published by M. Baratta*, A. Cavasino† and P. Caloi‡ in the Annali di Geofisica, etc.

In evaluating properly and correctly the seismicity of the area where a dam (together with other connected works for the different installations) is to be placed, it is necessary to keep in mind in the first place the two following considerations:—

(1) The assigning of the degree of seismicity for a whole region is at most an extrapolation from observations made in the areas where constructions and especially groups of houses and buildings were found destroyed to a greater or lesser degree.

The type of construction used in these buildings has a great influence on the observed effects of the earthquake.

Not seldom it happens that at a little distance of the centre of habitation destroyed to a large degree or totally by an earthquake, good constructions erected on geomorphologically more suitable ground would have felt the effects of the earthquake much less, and so would show little or no destructive effects of the same earthquake.

It is therefore improper to apply to a given locality of a commune what has been determined in its inhabited centre. It is essential that, before application, a suitable geological examination of the locality in consideration should be made (to be completed eventually with excavations, borings, and geophysical exploration).

(2) *Vice Versa*, in areas of the commune which show unfavourable configuration, the effects of an earthquake would have been seen as disastrous if the area contained houses or other constructions. The earthquake remained unobserved or nearly so because the area was uninhabited or remained without observation.

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*Baratta, M. : The earthquakes of Italy ed. Fratelli Bocca, Torino 1901. The earthquakes of Italy ed. Le Monnier, 1936.

‡Cavasino, A. : Earthquakes recorded in Italy in the thirty-five years 1899—1933 Appendix al Vol. IV & III delle Memorie del R. Ufficio Centrale di metereologia & Geofisica Roma, 1936.

‡Caloi, P. : Seismic activity in Italy in the decade 1930—1939 Vol. IX dello Pubblicazioni della R. Accademica d'Italia, Commissione di Studio periproblemi del soccorso allepopolazioni. Ed. le Monnier, Frienze, 1942.

15. *Conclusions*: The different points which we have explained in the preceding sections show that a very detailed and extensive examination of the locality where a barrage is to be sited is necessary; such examination should also be made of the localities where it is intended to construct the different plants, piezometric towers, etc.

It is on the basis of the results of this examination that the engineering calculations of the different works and of the dam itself should be carried out, taking into account the seismic stresses, which the preceding examination indicates are likely to occur.

For the dam itself the criteria given in this paper may be adopted. For the constructions overground, which are in general free of vibrations, account should be taken of the possible dynamical actions by having recourse to experiments on models. For the underground works we may even omit straightaway any consideration of seismic action because it is understood that the works will be located in the core of the ground or of the subsoil and placed at a sufficient distance from the peripheral region of unfavourable topography, that is of the formations likely to crack or undergo strong differential vibrations.

## APPENDIX

**The text of article 25, as approved by the Superior Council**
**Article 25—Seismic effects**

In localities declared as seismic of the first or second category, as defined by the laws in force, the construction of masonry dams is only permitted when the foundation rock has sufficiently uniform mechanical characteristics in addition to those prescribed in article 23.

In the seismic localities of the first category the calculations regarding dams are to be made taking account, in addition to the statical effects of the weight of the dam and of the water, of the respective dynamical effects, which in a simplified form may be taken to be equivalent to :—

(a) As regards the inertia of the masonry structure.

1. For upheaval shocks, an increase or decrease, of the actual weight of the structure of not less than 20 per cent.

2. For undulatory shocks, horizontal forces acting in any direction of not less than 10 per cent of the weight of the individual parts of the structure.

(b) As regards the inertia of water, a pressure uniformly distributed on the face of not less than 5 per cent of the hydrostatic pressure at the base of the structure.

The above-mentioned additional forces may be reduced to ½ of the values stated for seismic zones of the second category.

In reinforced concrete dams, the resistance to the resultant forces of traction, obtained after taking into account the additional forces mentioned above, may be borne by the metal reinforcement, and a verification of the respective forces acting on the concrete which covers the reinforcement, may be omitted.

For areas which are admitted as subject to earthquakes on the basis of their seismic records, even if they belong to communes not included in the catalogue annexed to the Royal Legislative Decree No. 2105 of 22-11-1937 and subsequent enactments, the designer may be ordered to observe the preceding rules; in such a case the area will be classified as of the first or second category, depending on the intensity and the frequency of the earthquakes which have occurred in the locality.

When the geological characteristics of the locality where the barrage is to be placed are found to be especially favourable, lower dynamical forces than those prescribed in the preceding paragraphs may be adopted, even if the locality lies in a commune classified as a seismic commune of the first or second category.

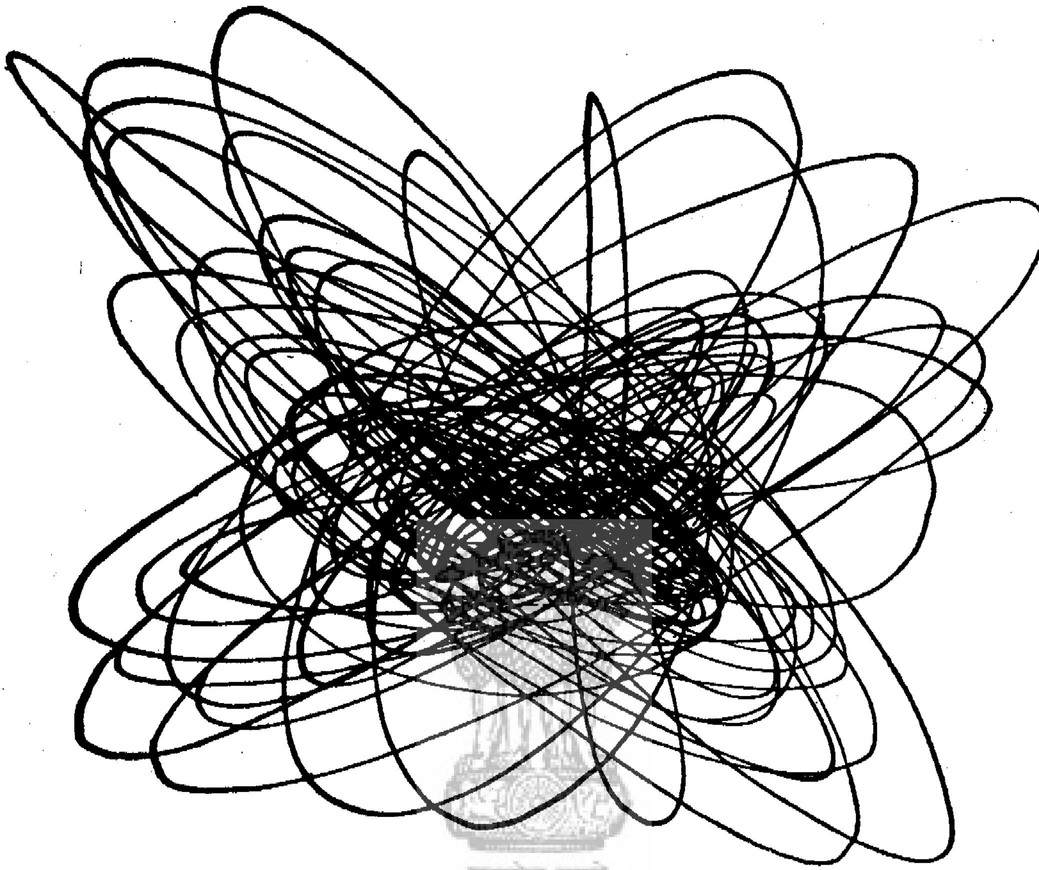



Fig. 1. Plane projection of the trajectory of a point in a formation subjected to an earthquake.

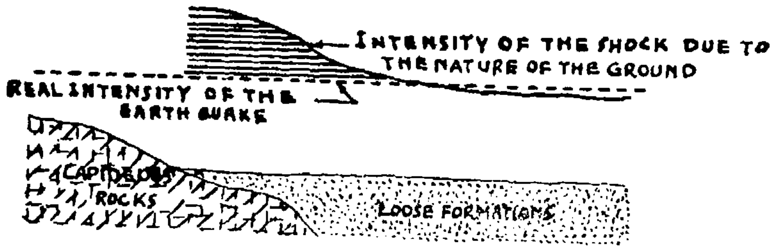


FIG:2 INFLUENCE OF THE THICKNESS OF A LOOSE LAYER ON THE INTENSITY OF AN EARTHQUAKE (FROM SIEBERG, 1933).

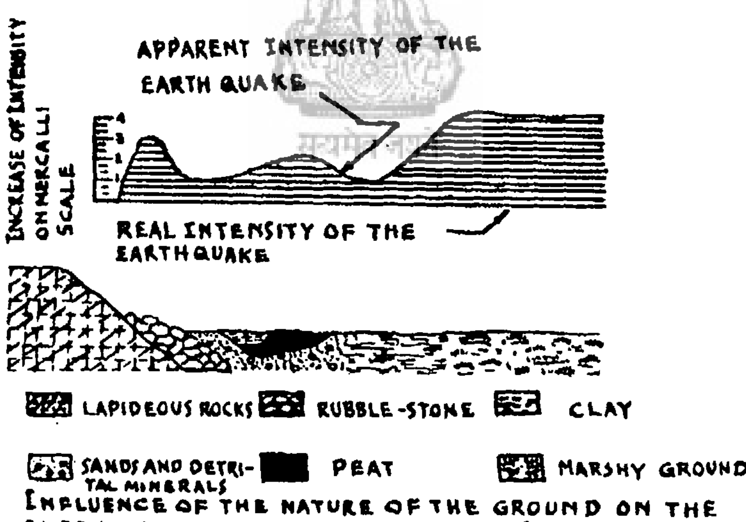


FIG:3 INFLUENCE OF THE NATURE OF THE GROUND ON THE INTENSITY OF A SHOCK AT THE SURFACE (FROM SIEBERG, 1933)

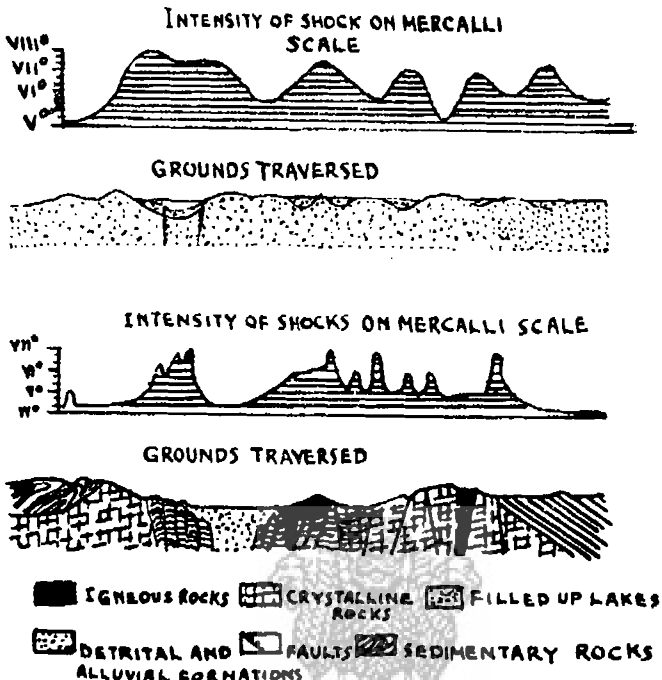


FIG: 4 INFLUENCE OF THE NATURE OF THE GROUND AND THE TECTONICS ON SEISMIC EFFECTS (COMPILED FROM THE EFFECTS OBSERVED AFTER THE CENTRAL EUROPEAN EARTH QUAKE OF 16-11-1911, ACCORDING TO LAIS AND SIEBERG).



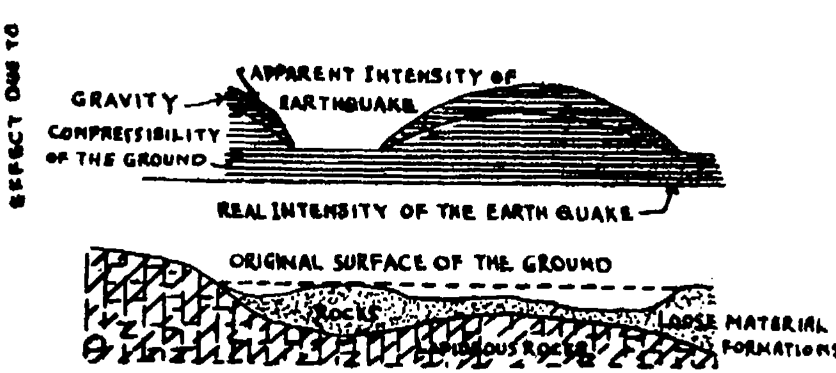


FIG: 5 INFLUENCE ON INTENSITY OF SHOCKS DUE TO MASSES LIABLE TO SLIDE OR SINK (FROM SIEBERG, 1933).

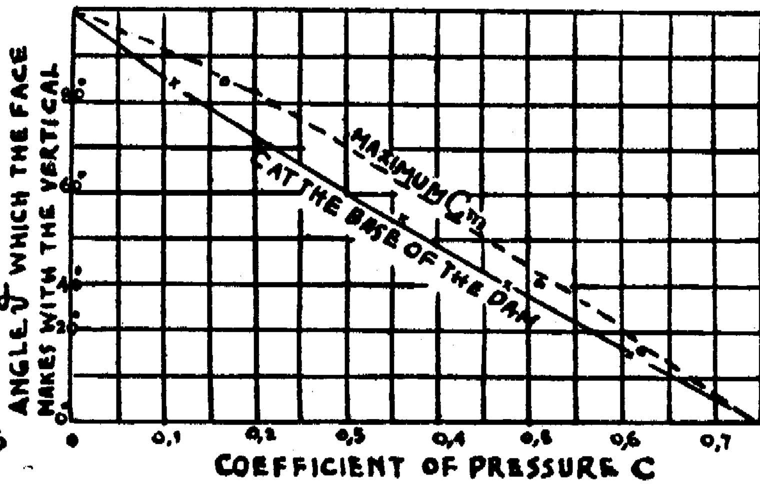
ANGLE ϑ WHICH THE FACE
MAKES WITH THE VERTICAL
MAXIMUM $C_m$
C AT THE BASE OF THE DAM
0
0,1
0,2
0,3
0,4
0,5
0,6
0,7
COEFFICIENT OF PRESSURE C

FIG: 6

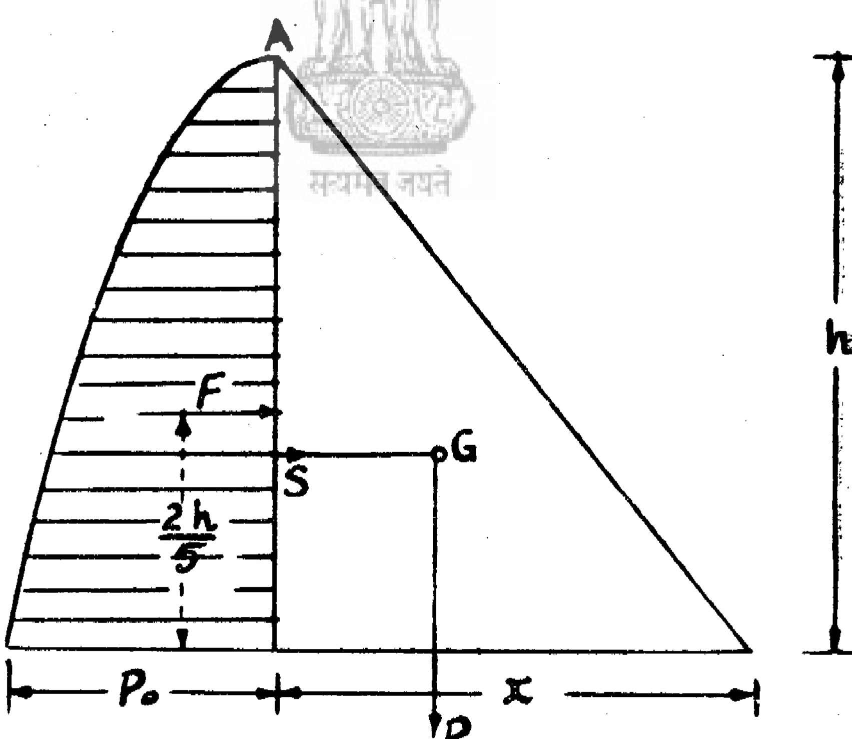
A
F
G
S
$\frac{2h}{5}$
h
$P_o$
x
P

FIG. 7

सत्यमेव जयते

# A CUPOLA-ARCH DAM IN A SEISMIC ZONE

## THE BARRAGE ON THE RIVER AMBIESTA AT CHIAICIS DI VERZEGNIS (UDINE)

### (1) General remarks

The Ambiesta Cupola-arch Dam has been built to form a barrage across a stream of the same name, which is a tributary of the Tagliamento in the region Prealpi Carniche. This region is highly seismic, classified as of category II according to the R.D.L. of 22-11-1937.

The dam has been constructed for the purpose of forming a reservoir to serve as a compensation reservoir, daily and weekly, for the hydroelectric works of Medio Tagliamento-Somplago. The reservoir is fed by a tunnel about 20 km long in which the water flows with a free surface; from this runs out another tunnel of 9 km length in which the water flows under pressure, conveying the water to the central Somplago which stands below.

### (2) Dates

Preliminary studies about this dam were commenced in 1949 and completed in 1954. The excavations were commenced in January 1955 and completed in November of the same year. This allowed the commencement of the castings immediately after that. Work was suspended during winter 1956, but the castings were resumed in the spring and finished in December of the same year.

### (3) Geology and Seismicity

The seat of the dam and its reservoir is an erosion furrow in the Dolomia called Principal of the Upper Trias. This rock is characterised by intense minute fracturation and marked fissurations which sometimes extend from one side to the other of the valley and which are themselves filled up with mylonitised rock. At the bottom, as well as in its interior at a certain depth, the rock is everywhere sound and does not show any longitudinal faults. Its strata are inclined upstream.

The region is intensely seismic and the shocks observed are frequent. Among the more severe recent shocks, we may refer to that of 27th March 1928, when there was a violent earthquake with shocks estimated at 9° and 10°, maximum acceleration of approximately 420 mm, and amplitude 50 mm, with the epicentre in a place near to the damsite (about 2-3 km distance from it). This earthquake resulted in cracks in builidngs, land-slips in the ground, clefts in the rock and a certain number of casualties.

It seems that the seismicity of the zone may have contributed to the fracturation of the rock, but it is believed that this fracturation is predominantly due to the intense orogenetic movements of the Alpine ridge.

This study of the project was made taking this circumstance into account (*vide* Section 5); the preliminary investigations were carried out with the cooperation of eminent seismologists and geologists.

First of all, we set up on the two banks, at a level corresponding to that of the arch of the crest, and above at a certain distance from the arch, four pairs of clinographs for the purpose of observing the slow movements of the rock. These movements were also studied with relation to the seismic phenomena of the zone, the latter phenomena being kept under observation by the use of seismological apparatus installed at Tolmezzo, at a short distance from the site.

There would have been great danger to the stability of the structure if the two imposts of the dam were to be found geognostically independent, but the preliminary investigations had for their principal purpose to confirm the singleness or sameness of the rock block comprising both the banks.

**(4) Reservoir**

The normal maximum level of water in the reservoir is about 484 m above sea level. The total capacity of the reservoir is 3,885,000 $m^3$, while the useful capacity which lies between the levels 484 & 455 m, is 3,600,000 $m^3$. The catchment area which directly feeds the reservoir is 9 $km^2$. The catchment area which served the dam in the first stage of the project was 311 $km^2$, and is 624 $km^2$ on completion of the project.

**(5) Dam—Characteristics, Calculations, Tests on Models**

The dam is of the symmetrical cupola type, built in concrete using pozzolana ferro-cement, weight 250 kg/$m^3$. It consists of a slab with double curvature which rests by means of a perimetral joint on a large "chaptrel". The maximum height from the foundation is 59.23 m. The total length of the coping is 144.64 m. The inclination of the face of the central bracket in the upstream direction is 60 per cent, and that in the downstream direction (on which side the jutting of the plumbline is maximum) is 48 per cent. The volume of the concrete used for the dam including that of the chaptrels is 28,734 $m^3$ and the volume of the excavation was 53,697 $m^3$. The planimetric radii of curvature vary on the intrados side from 29 to 71 m, and on the extrados side from 48 to 74 m. The chord of the mean arc of the crest between the chaptrels is 117.71 m, and the rise of the arch is 30.54 m. The minimum thickness taken at the summit under the arc of the crest is 2.1 m. The thickness at the base of the foundation chaptrel is 7.81 m, and that of the cupola is 5.52 m.

The intrados curves of the arches are 3-centered, and the extrados curves are circular. At the top the slab is made rigid by means of a powerful crest arch, in the central part and on this are sunk piles which support a platform for the roadway. The crest arch is founded on two small artificial abutments and contains a passage for inspection.

The reason why we selected a cupola-arch type, with strong inclination downstream, was because it was thought that this type of structure was most elastic and offered the best assurances of supporting exceptional overloads, as compared to gravity types of dam. Any other type of dam, like those built with dissolved materials, could not be adopted due to the circumstances of the locality and the difficulty of finding local materials. Even in case of a disaster, an arched type structure, with its various parts well-joined together, is considered to be the least subject to cracks extending throughout, even if the structure suffers considerable injury. This is probably because the various parts can remain in position, each part, as it were, supporting the other. Even if the injuries to the dam were such as to lead to the emptying of the reservoir, such cracks are likely to take place somewhat slowly, so as to minimise the risk of great rushes of flood downstream.

In particular, we designed a cupola with brackets (vertical sections) with a marked inclination downstream, as this would tend to lower the lines of thrust, and this would contribute much to the stability of the structure as a whole, even in the most unfavourable case of the seismic accelerations travelling from downstream to upstream.

Further, the resultants in this type of dam show a less tangential course as compared with other similar structures.

Still further, the dam, especially the part consisting of the chaptrel and the arch of the crest, is strongly fortified so as to permit it to resist and distribute any exceptionally strong stresses likely to be caused by seisms and the consequent possibility of small displacements of the foundation rock. Numerous strong stitches have been inserted, especially in places where the fissurations were most noticeable. All the voussoirs whether of the chaptrels or of the cupola have been joined with crossed iron pins, which form hinges and have also the purpose of sustaining the structure during construction, in view of the inclination to the plumbine of the central part. The total amount of iron used in the dam amounts to 5,327 q. (metric quintals) which is equivalent to over 18 kg. per $m^3$ of concrete.

The calculations and the tests for stability were carried out in accordance with the "Regolamento Dighe Italiano" (Italian regulations for dams), considering the dam divided into independent elementary horizontal arches subject to hydraulic load, temperature effect and contraction. Calculations relating to the arches were made by assuming that they are fitted into the imposts, and then applying the theory of elasticity. We tested also the stability of the central bracket considered as an element of small thickness of the rotation cupola. For all the brackets, besides, tests were carried out with the reservoir empty considering the cupola resistant or not to traction forces.

Finally, we carried out an analytical investigation of the seismic effects on the structure in acordance with the standards given in the respective Regolamento. Supplementary vertical and horizontal forces equivalent to 1/20 of the value of gravity were applied to the structure. The test was made when the reservoir was full and the dam subjected to a constant acceleration equal to 0.07 g in the direction of the chord and the rise of arch (*i.e.*, higher than the acceleration prescribed in the Regolamento), but taking into account that the tests did not exactly correspond to the conditions due to earthquakes. The forces resulting from the calculation would not exceed a compression of 52 $kg/cm^2$ and a traction of 7.4 $kg/cm^2$.

It was found that the dam undergoes considerable yielding at the imposts, before traction forces higher than 7 $kg/cm^2$ have been reached; more exactly, we have to admit a lengthening of the chord of 17.50 cm at the level of the coping but this is reduced to 4.375 cm at the level 450 m.

We also carried out the same tests on the model both as regards statical and dynamical stresses.

The maximum stresses observed during the statical test were found to be much lower than those calculated (29.7 $kg/cm^2$ instead of 56.4 $kg/cm^2$). This was mainly due to the effects of the rigidity of the arch of the crest and to the solidarity between arches and brackets; in part it was also due to the fact that in these tests we neglected the effects of the contraction and of temperature variations. In the breakage tests, the structure resisted well up to a stress 11.5 times higher than that due to the hydrostatic load. Up to the end the structure remained within the limits of the elastic state.

Of special interest were the investigations carried out on the model with respect to dynamical stresses. These will be referred to separately at this convention (Messina 1959) by Prof. G. Obesti, Director of the Institute for the Experimental Study of Models and Structures (I.S.M.E.S.) and his co-worker Engr. E. Lauletta. The tests at the I.S.M.E.S. were carried out in close cooperation with the authors of this paper. We shall give here a very brief account only.

First of all we arranged and studied the behaviour of a vibrating table placed on elastic supports, the model of the dam and of part of the reservoir being placed on the table. Every part of the model was constructed of special materials, such that there would be the best possible approximation between the breakage loads of the concrete of the prototype dam and the materials of the model. Even the specific gravity of the liquid in the model was suitably selected.

The vibrations were imparted by means of a pendulum giving damped sinusoidal vibrations or by means of a "vibrodine" giving vibrations of a constant character. The amplitude and the frequency of the vibrations could be varied at will. By means of measuring instruments, the characteristics of which were previously studied, we checked the inflections of various points of the model dam and the character of the vibrations imparted to it.

A partial breakage of the model dam (of the central upper part only) was obtained after numerous tests with damped vibrations imparted by our pendulum. These vibrations when converted into the values referring to the dam itself gave an amplitude of 168 mm; acceleration of 0.95 g and a frequency of 1.22 Hz.

The breakage when employing the "vibrodine" only took place after numerous and long-drawn tests, and only in the central upper part of the model, due to the effect of the persistent vibrations. These gave, when converted, an amplitude of 102 mm, acceleration of 0.92 g and a frequency of 1.5 Hz, after more than the first 35 minutes.

In both the tests referred to above, the oscillations produced waves in the direction of the chord when the reservoir was full (condition of maximum compression stress).

Still a third test, with persistent jerky shocks, led to a breakage (in this case also of the upper part only) after the first 50 minutes (converted value) with vibrations having amplitude 50.5 mm, acceleration 0.76 g and frequency 1.94 Hz.

In all the three tests, the seisms imparted to the model were of much greater range than those known, whether for the characteristics of the vibrations or for duration.

(6) **Fillings**

The consolidation and impermeabilization of the foundation rock demanded a large volume of perforations and fillings. In view of the constitution of the rock a special treatment was necessary to avoid the absorption of large quantities of the substance injected. We employed special materials for the washing of the holes and their lubrication (Silicates and the like), but the results were not very satisfactory.

The fillings did not lead to much improvement of the character of the rock; however, extensive investigations made by means of suitable borings gave us the happy assurance that, on the whole, the rock though fractured is markedly impermeable, compact and without cavities.

After the filling of the dam, losses of water were noted in the passages close to the imposts of the dam, especially on the right hand, upstream of the diaphragm. After a new series of fillings these losses were considerably reduced, though not eliminated.

Even the joints were filled in two stages, at the time of the principal contraction and at a low temperature of the concrete.

(7) **Instruments of Measurement and Control**

In the most characteristic points of the structure and the surrounding rock, we have placed various instruments of control, like thermometers, electro-acoustic extensometers, dialatometers and clinographs; additional to those normally set up for collimation and precision topographical survey. In view of the need for keeping under observation the seismicity of the zone, we have also installed in its neighbourhood, as already mentioned a 3-component seismological station. As regards the latter and other seismological and clinograph stations and their application for the checking of dams, a separate paper has been presented to this convention (Messina 1959) by Engr. Dino Tonini.

The following drawings and photographs illustrate the main characteristics of the dam described.

Fig. J-4—Ambiesta Dam —Upstream View

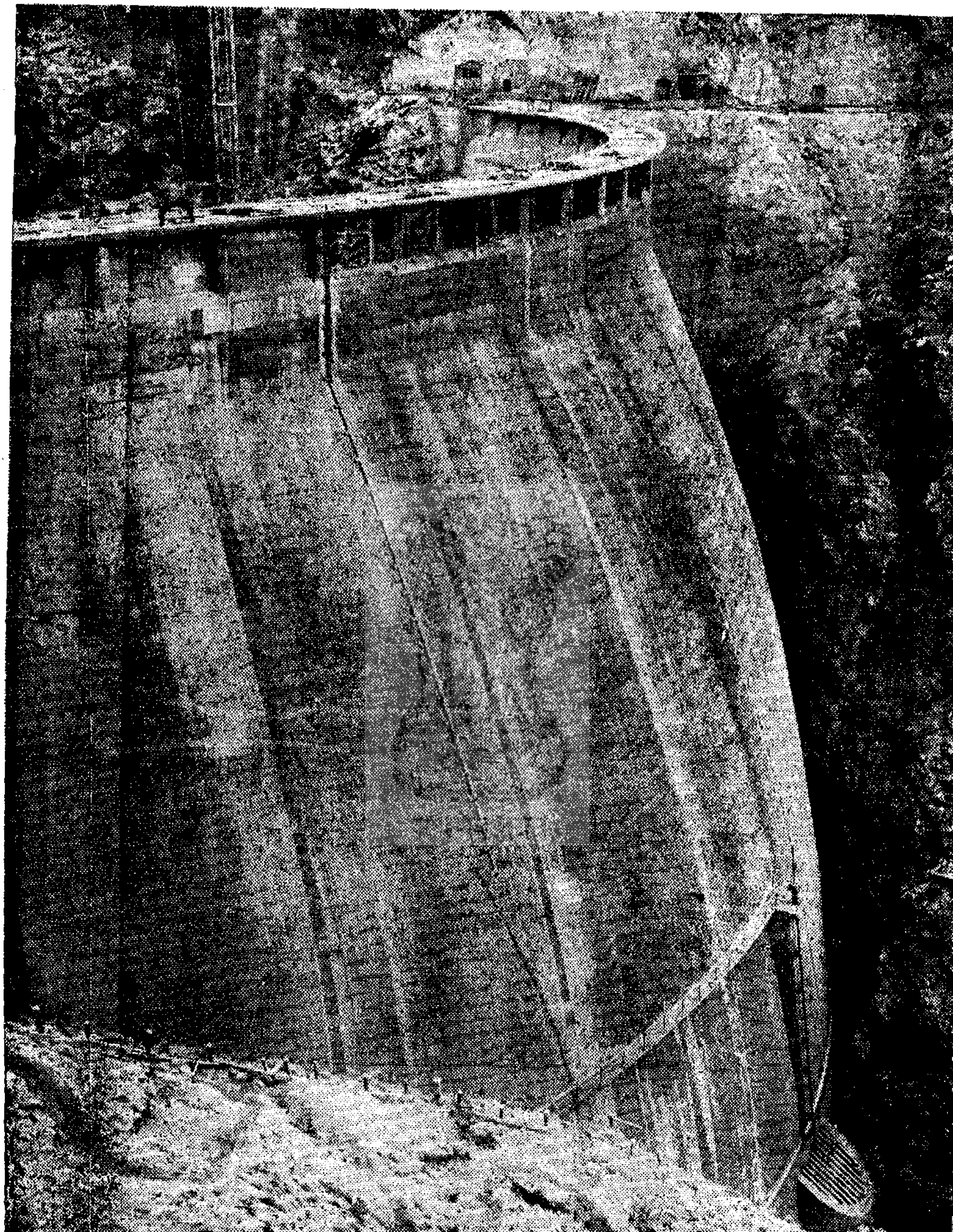

**Fig. J-5.—Ambiesta Dam—Left Side View**

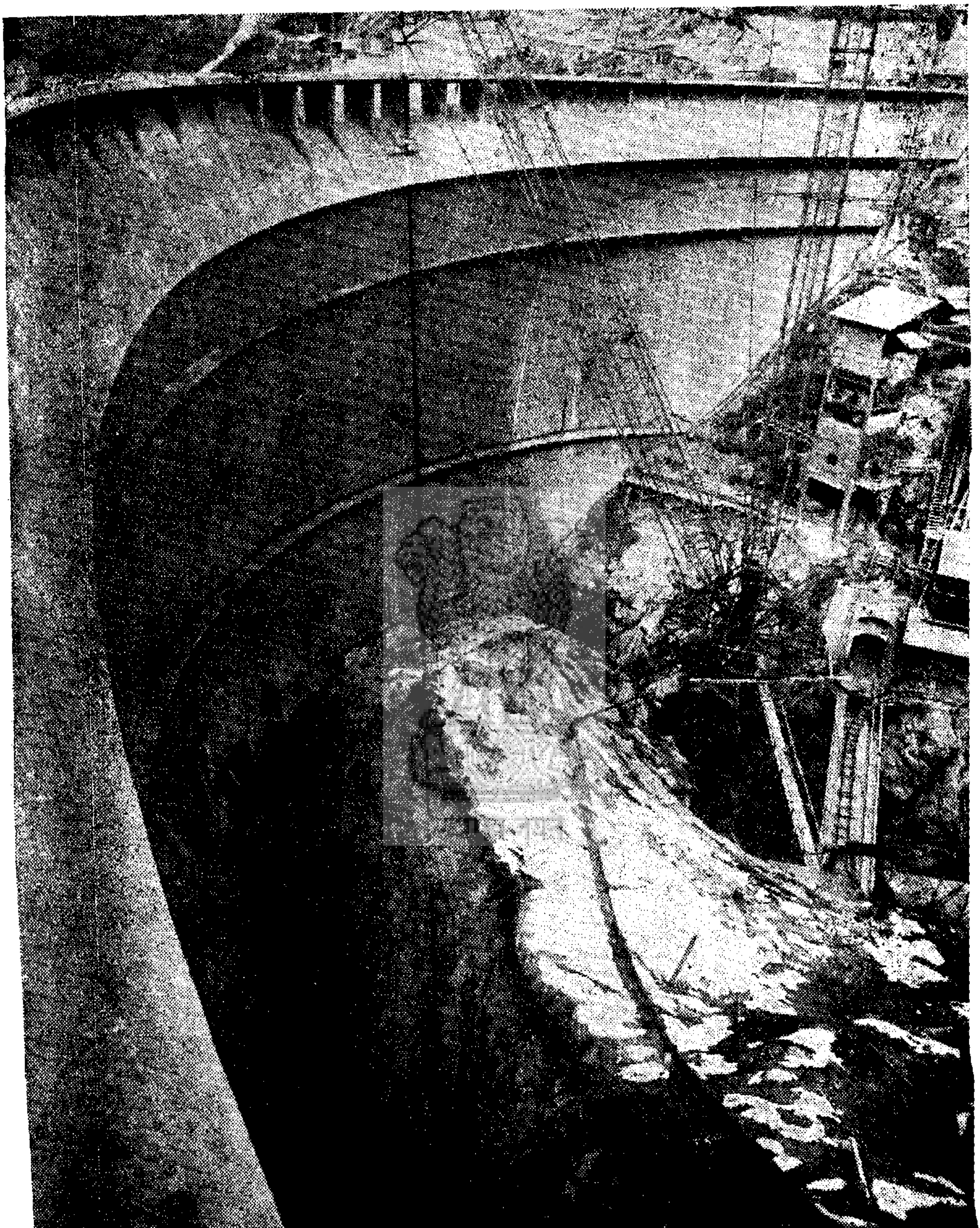

Fig. J-6.—Ambiesta Dam—Right Side View

## SEISMOLOGICAL & CLINOGRAPHIC OBSERVATIONS IN THE NEIGHBOURHOOD OF LARGE RETENTION DAMS

(1) Progress in science is only made possible by an ever increasing refinement in the methods of research : it is only by a gradual change in the investigational procedures, by the use of finer, subtler, more penetrating methods, that we are able to dive deeper and deeper into the mine of knowledge.

We should aim at limiting to the minimum if not at excluding altogether all direct observations (for our senses are fallible); we should use indirect observations, that is, substitute desultory personal observations by continuous instrumental records.

We should learn to enter into the core of a phenomenon, and not restrict our view merely to the more or less startling external manifestations. The latter are only the reflex of actions which have a much more remote origin, and it is not always easy to deduce the causes by the observation of superficial phenomena. We should, therefore, turn our attention from big oscillations or displacements to micro-oscillations or micro-displacements; from large changes of temperature, elastic stresses, pressure, etc., we should raise our mind to the examination of the micro-structure of matter (which is apparently at rest) and all its micro-manifestations.

Here lies the secret of every phenomenon, the origin of every external manifestation. It is quite apparent that the great disturbances originate : when they first begin to be perceptible, it is almost always too late to control them; it is necessary to seek them out in their pre-gestation phase, when everything is at rest.

(2) Hence arises the need, as already stated, of a continuous and minute supervision.



Let us consider, as an example, the slow movements of the earth's crust. Geodesy possesses the means for carrying out contours surveys and triangulations with extraordinary accuracy. But the geodesist undertakes his very precise survey work for some specific purpose only and in places and at times which are altogether unrelated; there are occasions when decades elapse before geodetic field work is undertaken a second time at the same place. For the purpose of a correct interpretation of the slow movements of the earth's crust, this type of survey is obviously insufficient. It is necessary to intensify observations in time and space; it is even necessary to find the means of replacing personal observations by continuously self-recording instruments. It is only by such a method that we shall be able to follow continuously the movements of a given region, and make it possible to watch, hour by hour and minute by minute, the department of a region which is apparently at perfect rest. That is the only way to detect the moment when the "microscopic" movements turn into "Macroscopic". If this moment is determined, we may be able to investigate how an imperceptible activity tends to become more and more perceptible. When we learn the *how* of the phenomenon, it will be easier to explain the *why*.

(3) Investigations carried out on this principle are also valuable in studies of smaller extension, as for example, in the study of the behaviour of a massive dam, erected for the formation of hydro-electric reservoirs.

The scientific interest is, in such a study, accompanied by distinctly practical considerations affecting human beings.

There are many factors which can endanger the stability of a dam.

It is not my intention to catalogue these factors or expatiate on them. I only intend to dwell on one type of the forces which can cause the most serious injuries to a dam—those released by the sudden movements of the soil which support it.

The engineer estimates the loads and the stresses which the dam may have to resist, but he is not in a position to counterbalance the intensity of the natural forces, when these occur in association with resonance phenomena.

All the techniques that human ingenuity can devise against earthquake risks prove insufficient if the ground on which the structure is erected is untrustworthy.

The solution of the problem, therefore, does not lie within the province of the science of construction above. It involves a whole series of relations of interdependence between the effects of the earthquake and the ground on which we build. It is necessary to acquire a complete knowledge of the geological and elastic characteristics of the surface structure of the earth, the predominating natural frequencies of these layers, etc., in order to determine, for each particular region, the type of dam which is most suitable for it.

Further, it is altogether insufficient to consider the problem solely from a statical point of view—not only insufficient, but erroneous as well. No doubt, the dynamical treatment of the problem is arduous; but we should not delude ourselves with the idea of having overcome the difficulty when, as is conventional, we replace dynamical actions by purely statical stresses, which are supposed to produce the same effects as the former. As long as the fundamental elements of the dynamical actions remain unknown, any supposed law of equivalence is a pure, unjustified extrapolation.

At this point I only draw attention to a problem which I considered at greater length in a previous paper (1). I shall return to the question shortly when treating of the damages which may be caused to dams by slow movements, the seat of which lies solely in the earth's crust.

Whatever be the angle at which the problem is viewed, there can be no doubt that a seismological station, fitted with three suitably constructed instruments, erected on the dam itself or in its neighbourhood, can provide extremely valuable indications about the behaviour of the dam. It is sufficient to emphasize the interest that attaches to a comparison of the vibrations obtained when the reservoir is full and when empty; or those registered in periods of drought and of prolonged spells of rain.

The slow movements of the basin—and hence of the voussoirs of the dam—are followed better by the use of clinometers. Considering that the elasticity of the rocks is not perfect and that their behaviour is influenced by other physical characteristics (like internal friction and viscosity), the slow movements are always accompanied by sudden movements of the type of small shocks. This is the reason for the great importance of seismological station at the damsite itself. The observations will show, when a given deformation occurs, whether it is elastic (independent of time) or a mass deformation (which is a function of time); in the latter case whether the deformation is temporary or permanent (for this purpose, obviously, the clinometer is the better instrument).

Also the study of the vibrations which affect the voussoirs of the dam are of notable interest. It is clear that these will present different characteristics depending on the level of water in the reservoir and the degree of tension to which the dam is subjected. These records will show a constant aspect if the elastic tensions of the dam do not undergo sensible variations, but in the case of a variation of the load, or an eventual crack, they will show different characteristics.

One thing is certain. No cracks, however small, can be produced in a dam, without their occurrence being promptly recorded in the seismographs.

With the use of seismographs and photo-clinometers the surveillance of dams becomes an automatic affair. Besides, the records provided by these instruments constitute irrefutable documents which can always be made to witness unto truth and justice.

**(4) Slow Movements of the Earth's Crust: Geodetic Blocks**

It is known that the earth's crust is the seat of slow movements of ascent and descent with respect to mean sea level; these movements may cause the emergence of more or less extensive land areas from under the sea, or lead to the submersion of continental lands adjacent to coasts. Obviously, such movements become manifest after very long periods and are known by the generic term of secular variations(2).

It has been proved in recent times that, on these movements of a general character which affect a vast region, are superposed local movements, specific to a limited area of the crust(2). The existence of the latter type of movements was shown by the frequent repetition of precision geodetic measurements in the same area. Clinographic observations are also excellent for this purpose. By such observations it has been found that sectors of the earth's crust, bounded by recent faults, are subject to block movements of ascent and descent, without any perceptible deformation of the blocks.

We think that this phenomenon assumes a special importance in question relating to the stability of dams.

Assuming the fact that the earth's crust is divided into blocks, it is extremely important to avoid the sitting of dams astride the zones separating two blocks.

Block movements of opposite sign, or even of the same sign but of different amplitude, may endanger the stability of a dam if it is situated between two blocks.

Methods exist which permit us to determine the boundaries of the geodetic blocks belonging to a given region.

One of the best methods is the use of photo-clinometers placed on both the banks of a river in the zone where the construction of barrage is proposed. The continuous records given by the instruments over a suitable period of time enable us to determine whether both the banks belong to a single geodetic block or each bank belongs to a different block.

(5) The research section of a big hydroelectric company which is under the direction of an eminent scientist, takes special interest in the importance of seismological and clinographic observations obtained in the neighbourhood of large dams.

In compliance with the request of this scientist, our institute provides the installation of suitable seismological and photo-clinographic stations near a large scale dam, of which the construction has just been completed.

Clinographic stations were later installed near other hydro-electric reservoirs of the same company.

We shall give here briefly some of the results obtained.

The detailed reports are in the possession of the company, who will examine the results with a view to their utilisation; these reports are, therefore, at present of the nature of confidential documents.

However, some characteristics of general interest may be deduced even now and published.

We shall begin with the seismological observations.

The seismographs were constructed in the workshops of the National Institute for Geophysics under the direction of the geophysist, Dr. Peronaci, and the instruments have mechanical self-recording arrangements. The principal instrumental characteristics are given in figures 1 to 3. The magnification was intentionally kept low since the shocks recorded would have their origin close to the instruments. At first, the three components (vertical, N-S and E-W) were placed close to the dam; and another instrument (N-S component) was installed at a distance of about 2 km.

At present, all the four instruments are functioning close to the dam.

On completion of the dam, as the reservoir was gradually filling up, records were obtained of very minute and slight shocks, originated by very slight vibrations in the rock system, the different portions of which tended to take up new equilibrium positions.

In many cases it was found possible to determine the point of origin of these small shocks—most of them originated at that part of the reservoir where the water had reached the greatest depth (of the order of 100 m).

Figures 4 to 7 give examples of the records of the slight shocks which originated in the reservoir.

Even during the first discharge from the reservoir, the instruments registered a considerable number of small shocks.

Many of latter had their origin at the base of the voussoirs and showed special characteristics (differentiating them from the former type), like greater frequency and the persistence of the vibrations *vide* figures 8, 9, 9 bis).

Some periods of specially marked microseismic activity observed, which were altogether independent of the filling up or the emptying out of the reservoir.

We shall speak later about this type of seismic activity.

It was possible to determine approximately the energy developed by each small shock, for four different types of shocks. Let us consider a simple harmonic wave of amplitude x and period T. The energy which crosses the unit area of a pencil of waves per unit of time is :—

$$U = \frac{\varrho}{2}\left(\frac{2\pi}{T}\right)^2 vx^2,$$

Where v is the velocity of propagation and $\varrho$ the density of the rock.

If we assume that the energy is propagated from the source uniformly in all directions, we can multiply U by $2\pi\Delta^2$, where $\Delta$ is the focal distance. This will give us the total energy crossing a hemisphere of radius $\Delta$ during unit time. Multiplying by the duration t in seconds, of the train of waves at the distance $\Delta$, the approximate energy of the shock may be expressed by :—

$$F_c = 4\pi^3 \varrho vt \left(\frac{X}{T}\right)^2 \Delta^2 \qquad \text{c. g. s.} \qquad (1)$$

In order to determine the value of $\Delta$, it is sufficient to know the interval s-p between the instants of the registration of the transverse waves (s) and the longitudinal waves (p). In previous researches we had already determined the velocity of propagation of longitudinal and transversal waves passing through the rock system constituting the bottom of the hydroelectric reservoir.

We may observe that p and s generally present a complete wave; the other vibrations that succeed s are to be attributed to reflections or to waves of a superficial type. Therefore, the value of t which occurs in equation (1) may be taken as equal to the period of the wave under consideration.

The result obtained by applying equation (1) to a single component, say, in particular to the waves s, would be divided by 4, since the waves sH (for which the vibrations occur in a plane normal to that of the propagation) are amplified by twice the free surface. The waves sV (for which the vibrations are in the plane of propagation) show, however, an amplification which is slightly less than in the former case. Nevertheless the result itself would be multiplied by 4, since the energy propagated by longitudinal and transversal waves is nearly equal, and further the mean energy in the spherical corona of the shock is equal to the mean kinetic energy. Therefore, the total energy for a single component is that calculated by applying equation (1) to the observed value of s (which is generally larger than p).

Four types of small shocks were considered, depending on the mean energy liberated, namely $10^{10}$, $10^{11}$, $10^{12}$, and 10 ergs. The number of shocks recorded with mean energy of the order of $10^9$ ergs was extremely large.

We shall refer later to the order of magnitude of the energy liberated during occasional periods of abnormal seismic activity.

At this point we wish to draw attention to another characteristic of the microseisms which is specific to the basin under observation.

It is known that, from the direction of the initial movements of earthquake, determined from a certain number of observatories suitably distributed around the epicentre, it is possible to deduce the physical nature of the cause of the earthquake.

Of what nature then are the small shocks observed in the hydro-electric reservoir under consideration ?

At first sight, a reply to this question may seem impossible; the data of a single seismological station are in face insufficient to explain the phenomenon in all its aspects.

However, in our case, a fairly satisfactory reply can be given. Let us recall that these small shocks were only observed during a very short period of time. The fact that during this short interval the seismological station recorded shocks characterised by compression or dialatation immediately excluded the possibility that the movements to which the shocks may be attributed were movements of ascent or descent. In fact, it cannot be admitted that vertical movements of opposite sign could occur at an interval of a few hours —or rather minutes—from each other, as in this case.

It is, therefore, quite logical to assume that the small shocks are to be attributed to slips *along small fractures*: The existence of initial signs of opposite sense for the longitudinal waves signifies that the plane of the fault changes from one shock to another, or from a series of small shocks to another. This interpretation is supported by the fact that the centres of the shocks are numerous, and therefore spread over the whole area across the dam.

(6) I have explained in another paper(3) the theory of the photo-clinograph of the type which is now installed near some of the dams. I shall only give here the formula that expresses the magnification of the instrument.

If $\delta$ represents the deviation of the image at the distance of 1 metre, we have :—

$$\delta = 75 \frac{g}{\pi^2 l} \psi T^2$$

Where g is the acceleration of gravity, l the reduced length of the bifilar pendulum, $\psi$ the angle of variation of the apparent vertical, and T the natural period of the pendulum.

The suspension filaments are made of costantana. The main characteristics of the instruments, which are operating near some dams and in a number of Italian observatories, are given below :—

| | |
|---|---|
| Modulus of rigidity of costantana | $= 6.1 \times 10^{11}$ dyn./cm$^2$. |
| Length of suspension filaments | = 9 cm. |
| Mass | = 16.5 g. |
| L (reduced length of the pendulum) | = 7 cm. |
| Diameter of the suspension filaments | = 75 microns. |

An example of the possibility of using photo-clinograph of the bifilar type for the registration of the variation of the apparent vertical is given in figure 10. This figure shows the record of the disturbances observed at the location of the central voussoir of a large scale dam which is under construction. They were caused by the coming and going of heavy cranes on the adjacent rails, which were laid parallel to the dam, in the space downstream from the latter, during the construction of the voussoir. The variations of pressure at the base of the voussoir caused by the displacements of the cranes were changed into variations of the vertical of the voussoir themselves.

Obviously such variations were almost exclusively recorded in the upstream-downstream direction. (Figure 10-11).

### (7) Records of the Diurnal Wave

The rays of the sun, when they shine on the exposed walls of a building, heat these walls causing thermal imbalances more or less marked, between the exposed walls and those in shade. The thermal action is accompanied by a mechanical action on the building. A clinograph placed inside the building would record any strong mechanical effect in the form of oscillations, the characteristics of which would vary from building to building and from locality to locality. These disturbances of the apparent vertical, which are associated with the thermal action of insulation, are known under the name of the "diurnal wave".

We should not fail to expect the recording of such diurnal waves within the voussoir of a dam.

Obviously, the intensity of the wave will depend on the exposure of the voussoir to sunlight. In some voussoir this exposure is very restricted, if not altogether absent.

The level of water in the reservoir has a notable influence on the diurnal wave.

When the reservoir is getting filled up, or is full, the diurnal wave is recorded distinctly—obviously, on sunlight days.

When the reservoir is dry, the diurnal wave is altogether absent, or is not perceptible.

The explanation of the phenomenon is as follows. The diurnal wave, as already stated, arises as the result of the thermal contrast between the different surfaces of the same structure; the stronger the thermal contrast, the greater will be the amplitude of the resulting thermal wave.

When the basin is full the thermal contrast on sunlight days is the greatest; in fact, the upstream walls of the voussoir will show a mean temperature of 4° to 10°; on the downstream side the walls may attain in summer temperatures above 40°. These are the conditions required for large diurnal waves.

If the reservoir is dry, the contrast will be much attenuated and, under certain conditions, it may be altogether annulled; this is the cause why only small diurnal waves, or no waves at all, are registered in this case.

The records obtained during the last phase of the discharge of a big reservoir (1951) are most interesting in this connection; figures 12-13 show the gradual re-appearance of the diurnal wave during the rise of the level of water in the reservoir.

The variety of diurnal waves observed is very great: waves characteristic of a given station, which themselves are different from month to month; almost perfect sinusoidal curves, or curves which are much deformed; symmetrical, or asymmetrical with respect to the "at rest" line.

When the daily thermal contrast is very notable, we obtain diurnal waves of large amplitude (e.g., 2 to 9 February 1952, figure 14). If the thermal inversion is sudden, we obtain maxima and minima which are pointed (e.g., 23 to 27 February 1952; 2 to 5 February 1952, figure 14). If the thermal inversion is gradual, the change of curvature of the maximum of the diurnal wave is also gradual (e.g., 14 to 17 January 1952, figure 15).

For small diurnal temperature changes we obtain curves of small amplitude. Even in this case the form of the wave is closely related to the temperature curve. A characteristic record is that for the week 15 to 22 December 1951, which shows sharp maxima due to sudden heating and thermal inversion, and on the other hand minima with large curvatures, due to the slow cooling (figure 16). From 22 to 26 December 1952 we have instances of days with short rapid heating and gradual long period cooling (figure 17). Other examples are found in figures 18 and 19.

Figures 20 and 21 relate to a smaller dam, of which the orientation to the sun was different.

Figures 21 bis and 21 ter show distinct diurnal waves superposed on long period (12-14 days) oscillations of considerable amplitude in the downstream-upstream component.

The indication of a photo-clinograph are particularly important in showing the behaviour of the dam during the periods of filling up and emptying out of the reservoir.

This behaviour, when other conditions are equal, is a function of the thickness of the dam and of the resistance (which may be expressed by means of the modulus of elasticity of the rock system in which the voussoir of the dam are embedded).

Figure 22 shows the course of the flexure of a large dam (during filling up of the reservoir) at a point corresponding to a specially massive voussoir (thickness about 36 m at the base); figure 23 shows the flexure of the same dam as seen by the effects in a voussoir of lesser thickness.

The flexure is generally very small.

Figure 23 bis shows the case of thrust downstream caused by a sudden increase of the level of water in the reservoir.

In the case of thinner dams, supported on rock systems with a relatively low modulus of elasticity, the amplitude of the flexure of the voussoir is obviously more perceptible.

Attention may be drawn to the instance reproduced in figure 24, which relafed to the operations of filling up and discharging of a reservoir formed by a relatively small retention dam supported on a bed of rock with relatively low modulus of elasticity.

Figure 25 relates also to the same dam.

As regards movements which cannot be attributed to changes in the water level or to sudden thermal or atmospheric variations, we may cite the case of the movements shown by the right shoulder of a large dam in the autum of 1950, a year after its construction.

The data relating to the water level in the reservoir and to air temperatures show that no external conditions were present to justify increases of stress on the dam and on the shoulders bounding it.

A brusque movement in approximately the NE direction which took place during the week 4 to 11 November, 1950 was followed—in the clinographic station of the right shoulder, placed on the rock—by the beginning of a large movement of about 90° in the succeeding week (11 to 18 November); there was a slight change of direction (N 42° W) in the week 18 to 25 November. During these 15 days, the angular movement was conspicuous (fig. 26). Simultaneously, the right side of the dam as shown by the clinographs of the voussoir nearest to the right shoulder No. XXIX—showed a slight flexure towards the rock formation at the right shoulder, which was retracing in the direction W 33° N ca. (fig. 27), that is in a direction very nearly the same as the longitudinal direction of the dam (S 42° W, fig. 27 bis).

This slight flexure of the right side of the dam towards the corresponding bank brought about a similar flexure of the central voussoir (XXIII, XIV) in the direction upstream-downstream accompanied by a slight component towards the right bank for voussoir No. XIII and towards the left bank for voussoir No. XIV (figs. 28 to 29).

The interesting feature in this case is that, about a year later (11 months, to be exact), the same phenomenon was repeated on the right side of the dam, namely a brusque flexure for which no cause related to the level in the reservoir or the air temperature could be attributed. The amplitude of this movement was smaller than that of in the preceding year. This marked the begining of an exceptional period of seismic disturbances as shown by the seismographic station functioning near the dam, namely on its right shoulder.

Was all this due to the movements of a block bounded on the east by the right bank of the river?

Indications like those mentioned above would lead us to believe that such a hypothesis may be accepted.

(8) A review of the clinographic observations in the period 6-5-1950 to 16-6-1951, between two consecutive discharges.

(a) Voussoir XIV (at the left centre of the dam).

The two graphs (fig. 29) refer to clinographic records obtained at the base of voussoir XIV in the periods 6-5-1950 to 26-8-1950 and 28-10-1950 to 16-6-1951. The interruption of about 2 months was due to disturbances caused by works which were being done near the clinographic station.

The diagrams give the resultants of the two records obtained simultaneously from the two components of the clinographs; one oscillating normally at the voussoir downstream-upstream, (N 10° E—S 10° W), the other longitudinally along the dam (E 10° S—W 10° N).

During the filling up of the reservoir there was a thrust downstream, which showed a nearly continuous course. This was followed by a period of relative rest, with minute contrasting movements.

From 28-10-50 to 3-2-51 the movements which continued to be small, and took place sometimes in one sense and sometimes in the opposite sense, were characteristic of a stable period.

From 3-2-51 the movements showed a decisive tendency towards the downstream-upstream direction, interrupted however by sudden partial reverses to the opposite direction.

From this time up to the time of complete emptying out of the reservoir, the graph shows an assemblage of figures similar to the Lissajous' figures.

As may be seen from the behaviour of voussoir XXIII and XXIX, the thrust in the direction downstream at the end of August 1950 could not be far from its maximum value.

The angular movement in the last three months preceding the discharge was, as may be noted from the figures, more perceptible than the thrust downstream caused by the filling up of the reservoir. We are led therefore to conclude at least as regards voussoir XIV—that, in its return to the no-thrust position, the voussoir passed across its original position and "bent" slightly towards the reservoir.

(b) Voussoir XXIII. In the case of this voussoir also, the angular movement was principally in the direction upstream-downstream and *vice-versa*.

The effect of filling up the reservoir was felt slowly and showed a distinct contrast up to 16-9-50 (fig. 29).

From the above date the effect assumed a definite and unindirectional character—excluding a few instances of the opposite sign in December 1950 and the first days of January 1951—up to 20-1-51. Beginning from this date, the direction of the movement was reversed until nearly the complete discharge of the reservoir.

In this last phase, the angular changes showed contrasts.

The diagram has been broken into two parts in order to avoid the inevitable superpositions of the lines which would otherwise blur the figure; the displacements during charging and discharging of the reservoir took place in fact in nearly the same direction.

(c) Voussoir XXIX. This voussoir showed a deportment which is clearly different from that of voussoir XIV and XXIII. In the first phase the angular displacement was in the upstream-downstream direction (with a slight deviation to the right) corresponding to the effect of filling up; this was followed by a period of rest, with a slight deviation towards the left (fig. 27). These displacements became more pronounced in September-October 1950. During the first days of November 1950 the angular motion took decisively a direction towards the right shoulder (fig. 27 bis.), coinciding with the general movement in this direction to which we referred in paragraph 7. This movement, which lasted till the end of 1950 was followed by a period of rest until the commencement of the movement which took back voussoir XXIX very nearly to the position it had at the beginning of the discharge (fig. 28).

(d) **Right shoulder.** The clinographs placed on the rock bed in the neighbourhood of the dam, on the right shoulder, showed a somewhat peculiar behaviour. The angular movement in the first months after filling up the reservoir took place mainly in the SE quadrant (fig. 26). During the first days of November, coinciding with the similar movement observed in voussoir XXIX, there appeared a decisive angular motion in the mean direction S 42° E—N 42° W, which was of exceptional amplitude in the weeks 11 to 18 and 18 to 25 November 1950. On 24-2-51 the movement was reversed, and on completion of the discharge, the apparent vertical of the site at the left shoulder was found perceptibly displaced from the apparent vertical at the beginning of the filling up of the reservoir.

(e) Summarising, the behaviour of the voussoir in the period between one discharge (1950) to the next (1951), was as follows: Voussoir XIV completed a to-and-fro angular movement very nearly in the same direction (upstream-downstream and downstream-upstream), but with a slight component towards the left shoulder. In the return movement the voussoir overpassed slightly its initial position, "slanting" towards the reservoir.

Voussoir XXIII showed a similar behaviour, that is angular motion almost exclusively in the direction upstream-downstream, downstream-upstream, but with a slight component towards the right shoulder. After the 1951 discharge this voussoir was found very nearly in its initial position.

Voussoir XXIX, after a slight displacement in the direction upstream-downstream, showed a distinct angular displacement towards the right shoulder. This voussoir was also found, at the end of the second discharge, in very nearly its original position.

The strong component of the displacement towards the right shown by voussoir XXIX is explained by the fact that this voussoir is near to the right bank of the river; in view of the form of dam (gravity arch), it is clear that a central thrust in the direction upstream-downstream involves a perceptible component towards the banks for the lateral voussoirs.

The clinographic station placed on the rock formation at the right shoulder showed the movement which we have described in (d) above. At the end of the second discharge the apparent vertical was found to be quite distinct from the original direction. It follows therefore that the rock formation on which the clinograph was installed underwent a permanent deformation, though, maybe of small amplitude. This explanation seems obvious. While the concrete of the voussoir is a compact material, the left shoulder on which the dam is supported consists of fissure rock full of veins of water. Under the increased pressure during filling up of the reservoir, the small cavities get contracted and this prevents the rock formation from its primitive position when the pressure is reduced.

(9) Comparison between clinographic and seismographic records :—

The question may be raised whether the variations in intensity and frequency of the small seismic shocks recorded have any relation with the similar variations in the run of the clinographic records. Generally there is close relation between the more or less rapid variations of the apparent vertical and the number and intensity of the recorded shocks.

The variations of the apparent vertical correspond to flexures in one sense or the other of the voussoir of the dam between which the clinographs function.

I could cite numerous examples of a surprisingly close correspondence between abnormal seismological activity and abnormal clinographic activity.

I shall only adduce a few of the most significant instances.

The notable seismological activity, both as regards the number and intensity of shocks, which occurred from the first days of February to May, 1951 is closely associated with the slow movements of the voussoir recorded by the clinographs from December 1950 to February 1951 the movements of the voussoir were extremely small. In the first days of February 1951 there commenced suddenly a series of comparatively rapid angular variations, with a general tendency towards the downstream-upstream direction; these variations were especially marked on the left side of the dam (as may be seen from the clinographic record of voussoir XIV (fig. 30).

It is remarkable that these bending of the voussoirs towards the reservoir, which was in the process of discharge, did not occur as a continuous movement, but alternated with reversals of direction; this was particularly so in the case of voussoir XIV. It is just during this period that the number and intensity of the shocks was particularly noticeable. The second series of shocks analysed by us refer exactly to the beginnings of February. There is no doubt that the enhanced seismological activity should be attributed to the complexity and amplitude of the movement of voussoirs, which in their rapid motion to-and-fro created in the subsoil the conditions favourable for small fractures and subsequent slidings along these fractures.

Let us note further that during the period mentioned, voussoir XXIII—and presumably the neighbouring voussoir as well—showed a whole series of rapid movements superposed on those of a longer period (fig. 30). Naturally these movements caused vibrations in the rock formations into which the voussoirs are embedded, and consequently induced sudden stresses, from which arose the series of small shocks.

Another characteristic period of exceptional seismological activity is that associated with the period of clinometric movements which commenced in the first days of October 1951. These movements began almost suddenly on the 6th October in the form of a slight flexure in the downstream direction of the voussoir on the right half of the dam; this may be seen from the clinograms of voussoirs XXIII, XXIX and that on the right shoulder, where the images were thrown off the record (fig. 32). It is precisely, during these days, and coinciding perfectly in time, that we observed some of the most exceptional records of small shocks (figs. 9, 33, 34), which had a very special appearance, distinguishing them from shocks of a different origin. The clinometric movements, due to similar movements of the voussoir, began again towards the middle of October and lasted, almost without intermissions, up to the end of 1951.

During the same period of the intense seismological activity continued unabated, testifying to the close association between the slow movements of the voussoir and the small and sudden seismological movements (figs. 35, 36, 37).

This is the proper point to draw attention to the fact that the small shocks recorded show a different aspect, depending on their origin. If their epicentre is within the reservoir, the seismic waves, even of considerable amplitude, are rapidly damped and so comprise a small number of oscillations. If on the other hand the shocks originate at the base of the voussoir close to the seismological station, they consist of a long series of very rapid oscillations. This can be explained by the fact that the concrete of the voussoir (as was confirmed by experiments in the Lumiei Dam) conducts and enhances the very rapid vibrations, in regard to which the voussoirs show a deportment somewhat similar to that of the prongs of tuning fork.

(10) **Movements of diverse origins.** The photo-clinographs show a large variety of movements—rapid, slow, periodic, or aperiodic—the origin of which is not always easy to determine. A few examples are given below:

On the 31st December 1950 the upstream-downstream component of voussoir XXIII of the large dam (to which most of these observations relate) was registering a slow and long period oscillation corresponding to a slight increase of pressure on the dam caused by the water in the reservoir. Towards 19 hours of the same day the clinometer of the same component (fig. 38) showed a sudden movement towards the direction corresponding to a decrease of the pressure. This was the beginning of a whole series of rapid irregular oscillations, among which one finds even uni-nodal oscillations (of about half-an-hour period) mixed with other having periods of 1 hour or more. Towards 15.5 hrs. of 1-1-51 there was a sudden movement of the clinometer in the direction upstream-downstream; the rapid irregular oscillations continued till about 18 hrs. of 2-1-51.

The curve of the atmospheric pressure was altogether normal; the phenomenon observed cannot therefore be attributed to the latter, except perhaps if a train of micropulsations in the atmosphere (not registered by the barograph) caused some sort of vibrations in the mass of water in the reservoir. The wind was blowing from NNW, with moderate velocity. The clinographic movements cannot however be attributed to the wind for two reasons : First because, when blowing from the direction mentioned, it should have caused an accumulation of water towards the dam; secondly because winds of that velocity blowing from that direction are common, but do not ordinarily cause the type of movements referred to.

We have numerous other examples of such movements, some of them still more startling than the above.

I shall mention another case of records, also obtained in the same voussoir XXIII, because the clinographs attached to it give slow moving records which permit a good resolution of rapid oscillations. The month of February 1951 was specially remarkable for both slow and rapid disturbances. Attention is invited to the tracing reproduced in fig. 39, and more specially to that reproduced in fig. 40, where the rapid irregular movements are indeed most notable.

We frequently find the simultaneous occurrence of irregular vibrations of very long period (8 to 10 days), which last for months. Occasionally such vibrations seem to be related to similar temperature oscillations (e.g., voussoir XIV); but such parallelism does not in general exist (vide for example 8-1-51, voussoir XXIII); it seems therefore more logical to attribute these disturbances to oscillations of the geodetic block supporting the section of the dam where the movements are observed.

The photo-clinographs register also the small variations of the apparent vertical caused by the free oscillations (sesse*) of the lake: The alternation of the maxima and minima in the amplitude of the sesse determine small variations of the pressure at the bottom; these variations are transformed by the dam into small variations of the apparent vertical. We are obviously referring to clinographic oscillations of small amplitude, of period differing largely from the natural period of the voussoirs themselves. The uni-nodal sesse of the lake has in fact a period of the order of 25 minutes (figs. 9—11).

Another type of record has been noted near the seismological station of the large dam; this type of record is mainly of geophysical interest.

Among such records we shall first mention the very numerous and extremely rapid oscillations recorded in hundreds on certain days of December 1951. These oscillations, which taken all-in-all have the appearance of small spindles, consist of such rapid vibrations that it is not possible to resolve them even after magnification. I have no hesitation to attribute these vibrations—recorded when the seismological station was provisionally stationed on the shore of the lake—to small shocks communicated to the shores by the cracks which, on certain days, occur in the ice of the frozen lake; it is well known that the vibrations which originate in ice ("Biegewellen" in German, "flexural waves" in English) are generally of a very high frequency.

Another phenomenon of a vibratory character and of great geophysical interest are the records of very minute microseismal waves which last for weeks, and the cause of which is unknown. It

*Sesse—Vertical oscillations of the water of some large lakes, independent of lunar-solar causes—Translator.

is well known that the term "microseismic movements" is employed to indicate only those disturbances, periodical or otherwise, in which only the surface layers are involved and which therefore are not, properly speaking, of seismic origin. The microseismic agitations which have been studied in greatest detail both as regards their scientific and practical aspects, are those associated with atmospheric disturbances, and specially those associated with the passage of cyclones over the oceans and their respective inland seas. Such agitations, consisting of groups of oscillations of periods ranging from 1 to 12 seconds, depending on the distance of the focus of disturbance and the geological characteristics of the strata traversed, are registered by all the seismological stations of the world—sometimes more clearly, sometimes less clearly; these oscillations are most noticeable in winter.

The type of vibrations mentioned in the closing lines of the preceding paragraph is not the same as the type of vibrations recorded by the seismological stations attached to dams. Nor can the latter type of vibrations be attributed to traffic, in view of the great distance of large cities, or trunk roads, or railways, and also of the conditions under which the vibrations occur. I have the intention of giving my attention to this problem with a view to discovering—if possible—the causes of these very minute vibrations of dams (those illustrated in figs. 42, 43 and 44 show a period of the order of 1/10 sec.). Are they due to minute orogenetic movements? This is what we hope to clarify by pursuing special investigations for the study of these minute vibrations.

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ROME, NATIONAL INSTITUTE OF GEOPHYSICS, JULY 1953

**Summary**

Some of the results obtained in a first period of geophysical observation (in particular, seismic and clinographic) of a vast retention dam for a hydroelectric reservoir are briefly explained here. From the time that these results were first made known, the interest has been clear in such continuous observations. Of scientific interest are the following: Any new information that one can extract with respect to the behaviour of a large dam, the comparison of the rocks to which it is secured, and knowledge of the forces to which it is subjected on the part of the reservoir, and also that many perturbing causes have no relation to the science of construction, being of merely geophysical origin (such as the movements of geodetic crustal blocks). Of practical interest, not less important because they give a continuous, prompt, and very sensitive means of control, are the measurements given by special seismographs and photoclinometers placed near a great dam. Other measurements on minima and flexures can quite possibly be carried out with the avoiding of large scale damage.

**Bibliography**

(1) Caloi P., "Seismic Activity in Italy in the Decade 1930-39.

Appendix :—Seismic Buildings and the Reactions of Buildings and the Subsoil to Seismic Stresses". Felice li Monnier, Firenze 1942 (137 pagg., 42' figg nel testo e 72 tev. fuori testo).

(2) Caloi P., "Geophysical Interpretation of Geodetic Measurements".

Annali di Geofisica, IV (1951).

(3) Caloi P., "The Horizontal Pendulum used as Clinometer".

Annali di Geofisica, III (1950).

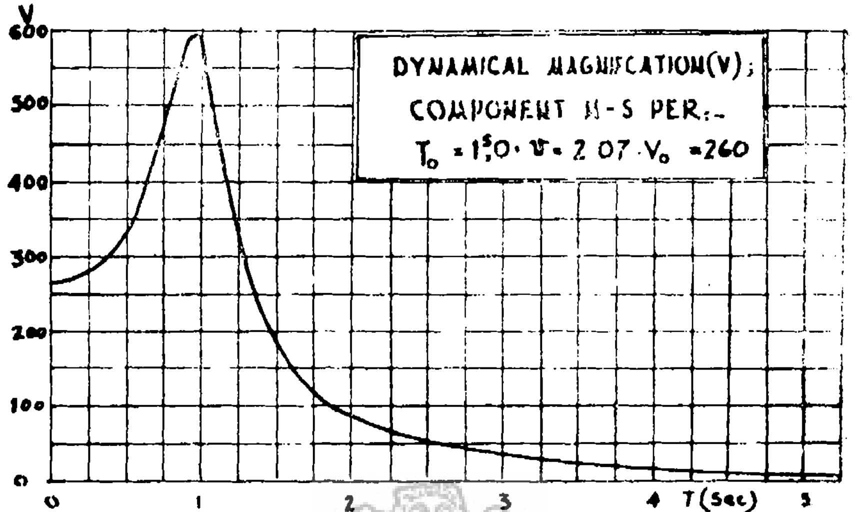
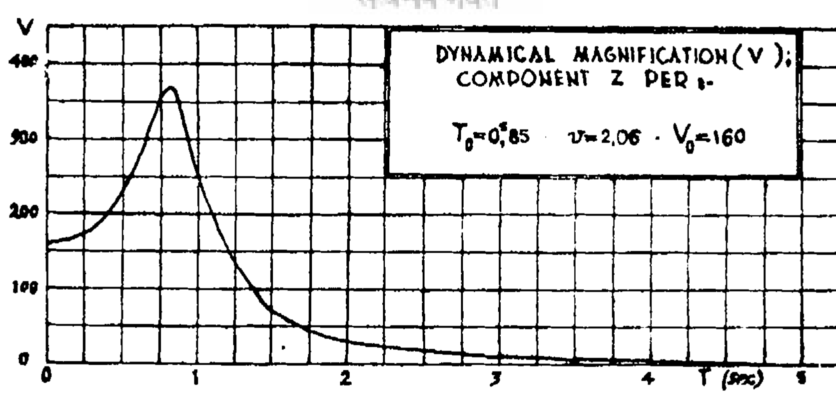

DYNAMICAL MAGNIFCATION(V);
COMPONENT N-S PER:-
$T_o$ = 1,0 · ν = 2 07 · $V_o$ = 260
V
600
500
400
300
200
100
0
0
1
2
3
4
T (Sec)
5

FIG. 1




DYNAMICAL MAGNIFICATION (V);
COMPONENT Z PER:-
$T_0$ = 0,85 ν = 2,06 · $V_0$ = 160
V
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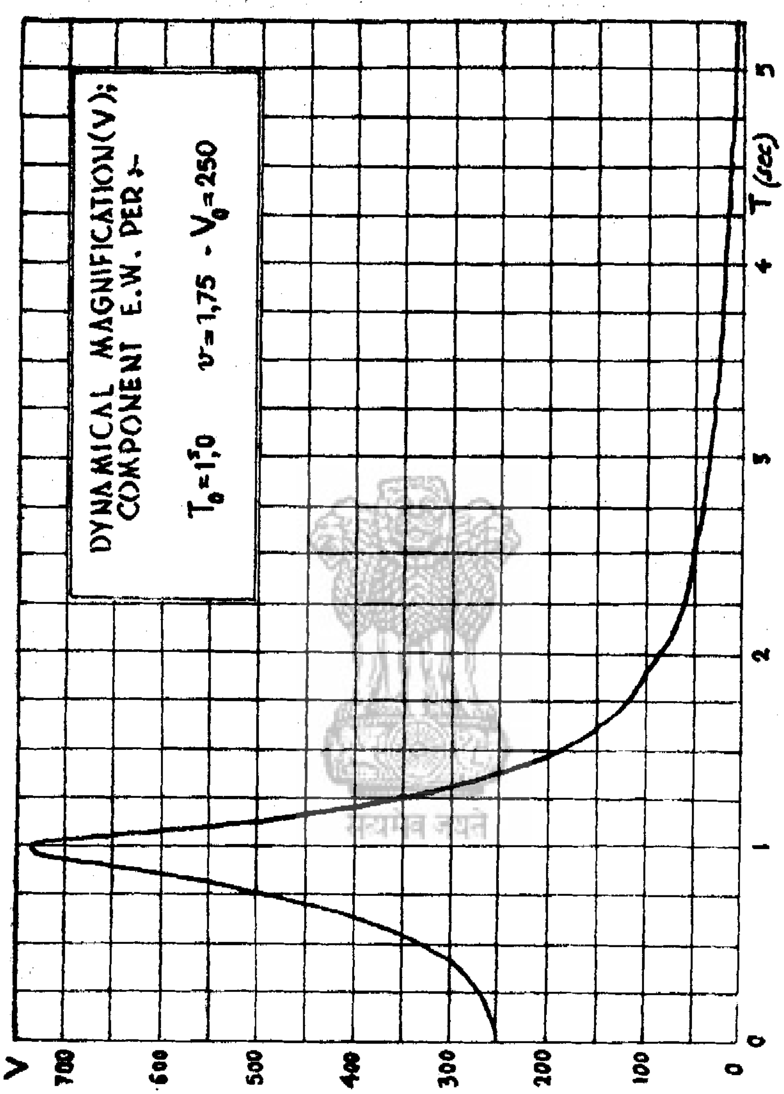
300
200
100
0
0
1
2
3
4
T (Sec)
5

Fig. 2


DYNAMICAL MAGNIFICATION(V);
COMPONENT E.W. PER.;—
$T_0 = 1^s,0$
$v = 1,75$ - $V_0 = 250$
V
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600
500
400
300
200
100
0
0
1
2
3
4
5
T (sec)

Fig. 9

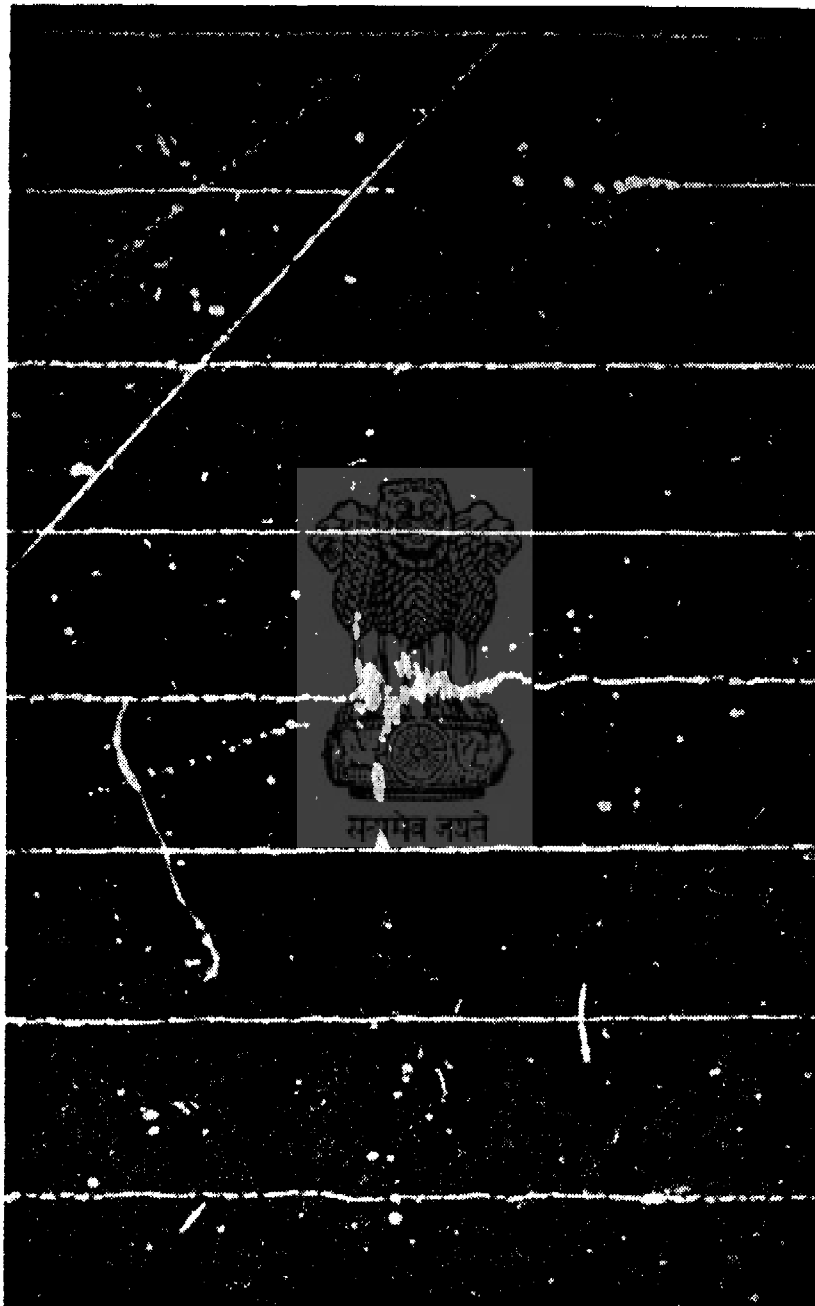


Fig. 4—Small shock estimated to be of degree III ($E=2.10^{12}$ erg. approx.) Magnification about 12 times the original

Fig. 5—Small shock estimated to be of degree II (E=3.10$^{11}$ erg. approx.)

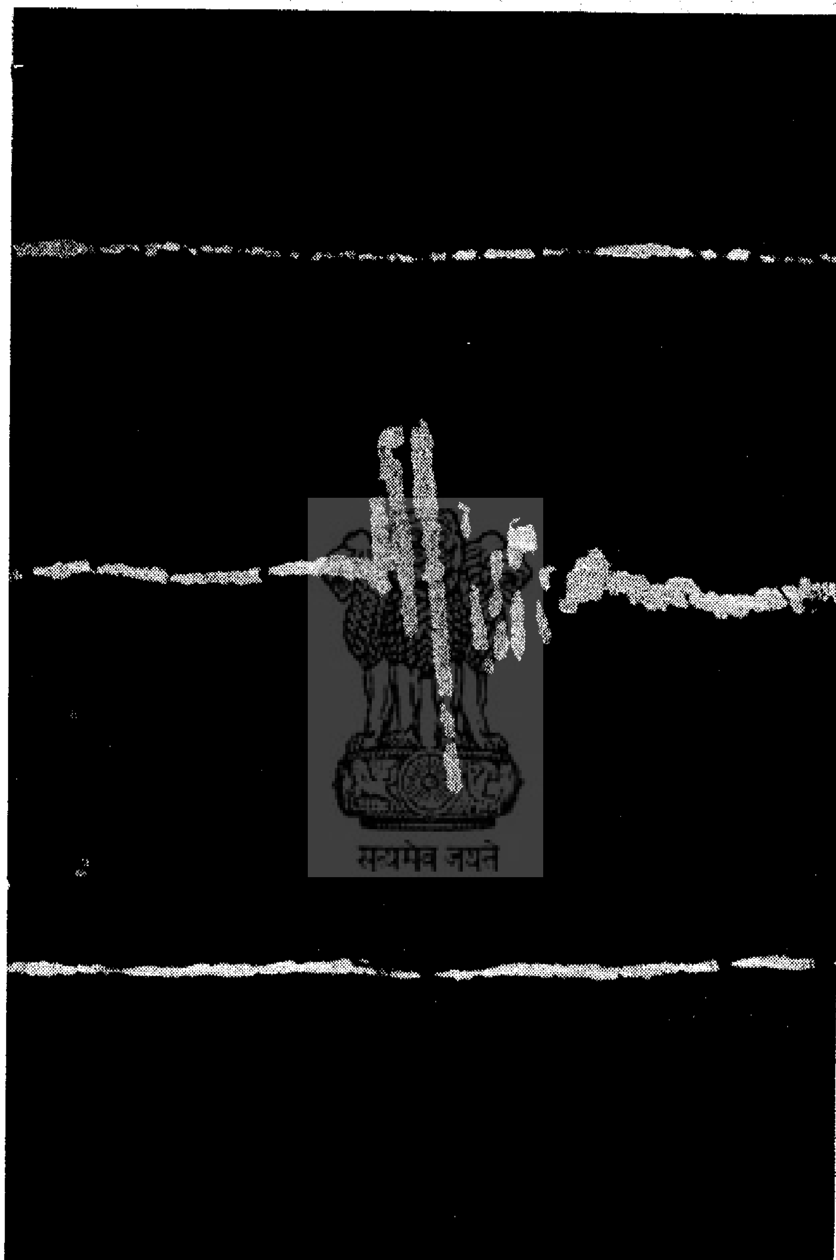


**Fig. 6—Small shock estimated to be of degree II (E = $10^{11}$ erg. approx.) Magnification about 25 times the original**

Fig. 7—Small shock estimated to be of degree II ($E = 1.5 \times 10^{11}$ erg. approx.)

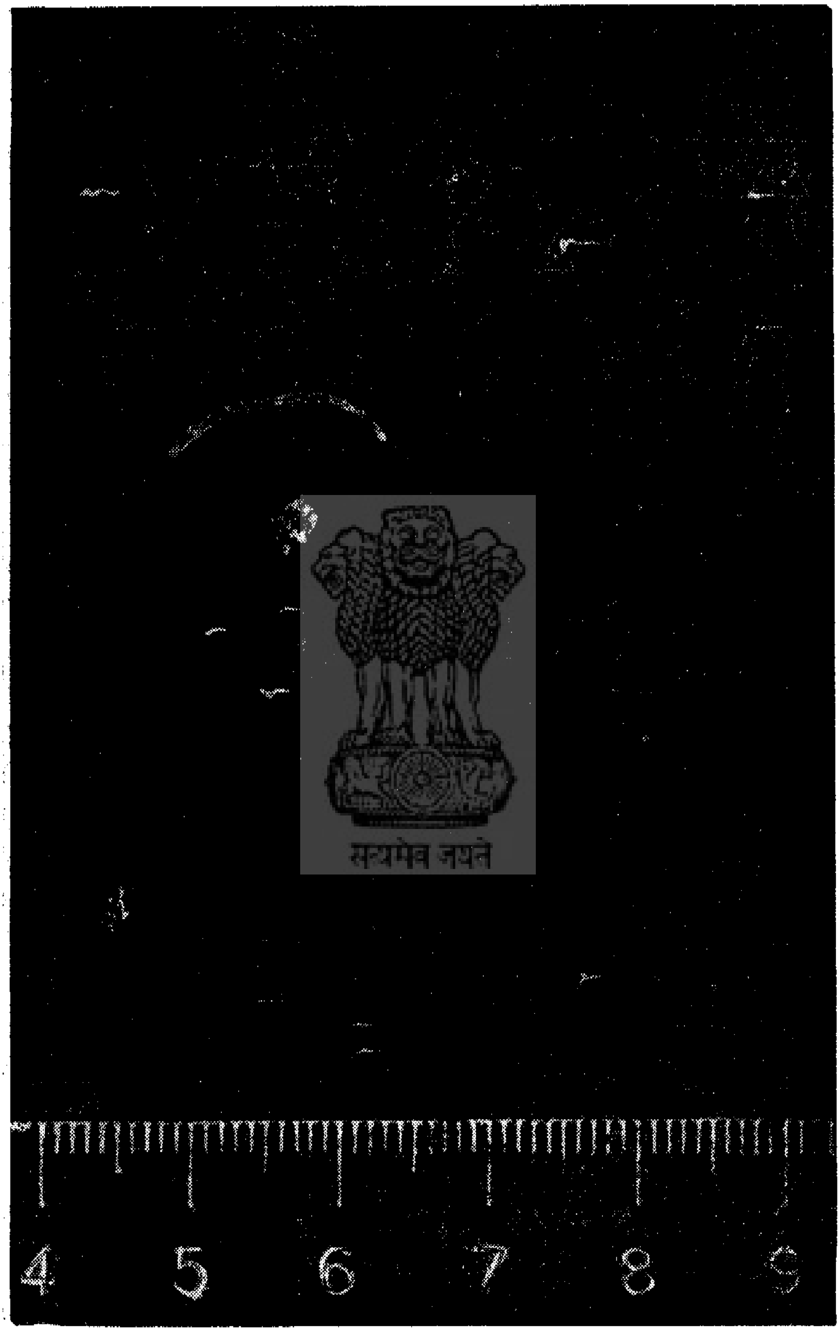



**Fig. 8—Small shocks transmitted from the voussoirs in the neighbourhood of the Seismological Station (Component E-W : 2-3 Nov. 1951)**

Fig. 9—Very intense Seismic activity (Component E-W : 9-10 Oct. 1951)

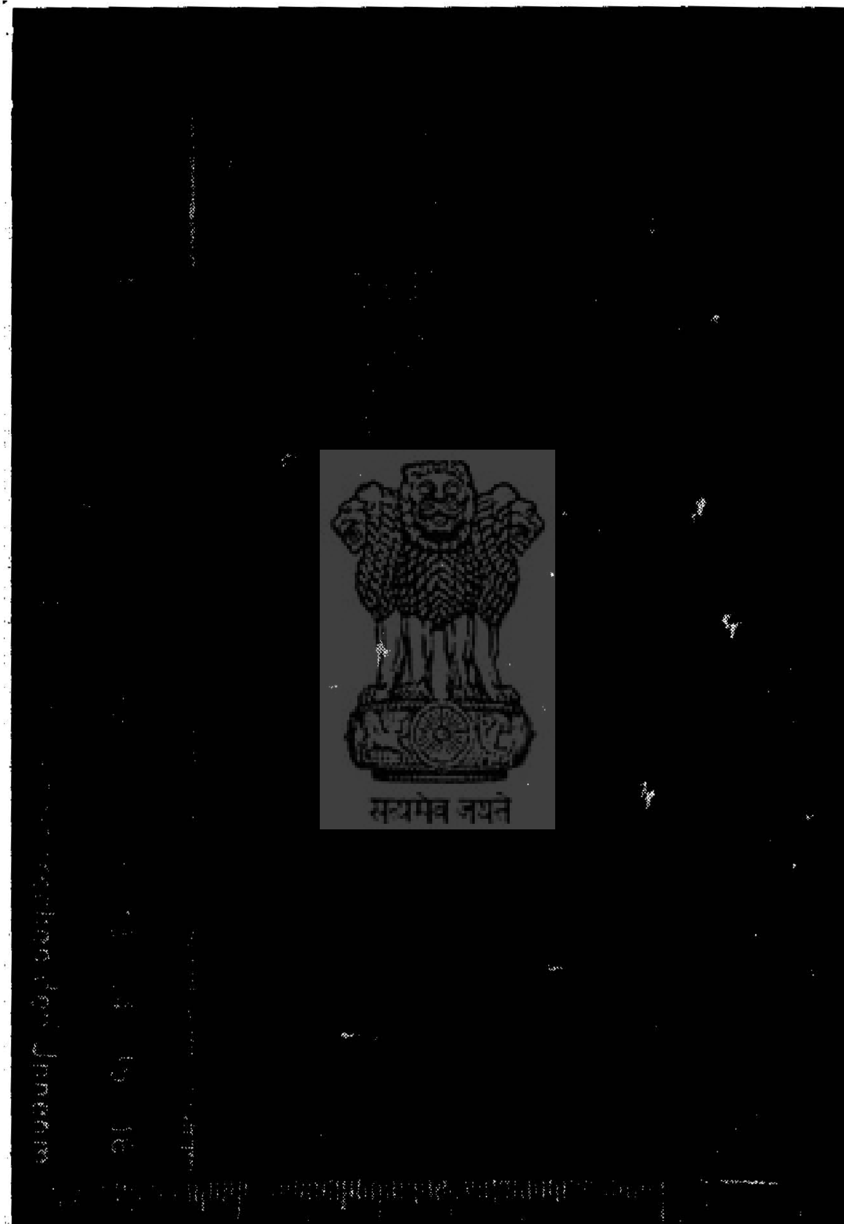




Fig. 9 bis--Intense activity (Shocks of degree II & III of 12-13 Nov. 1951, Component E-W)

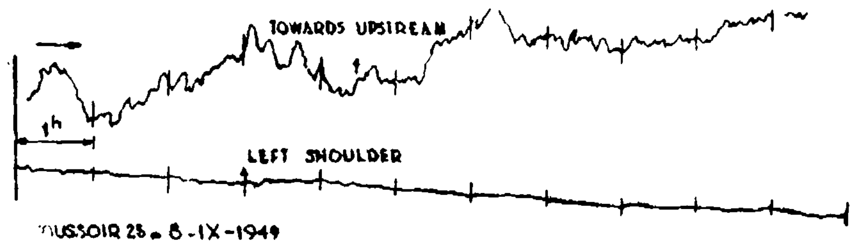



Fig. 10

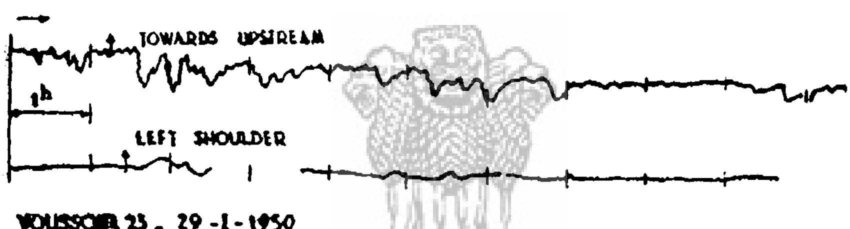



Fig. 11

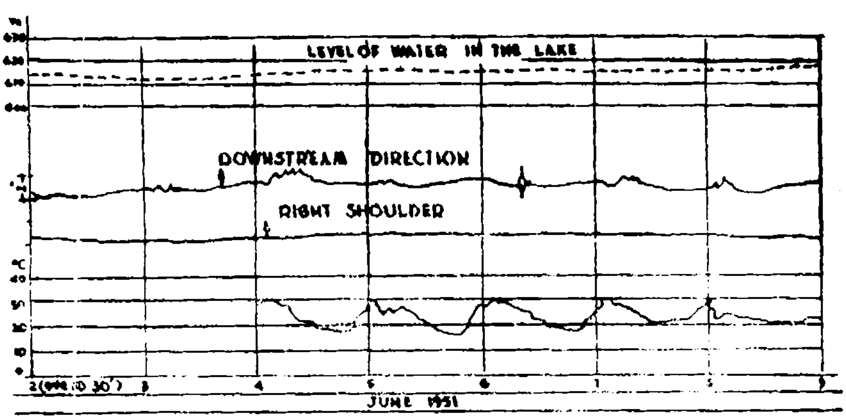



Fig. 12

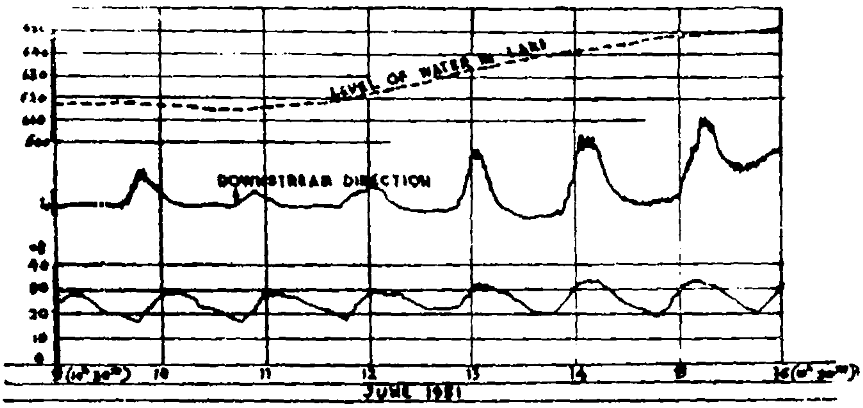

LEVEL OF WATER IN LAKE
DOWNSTREAM DIRECTION
JUNE 1951

Fig. 13

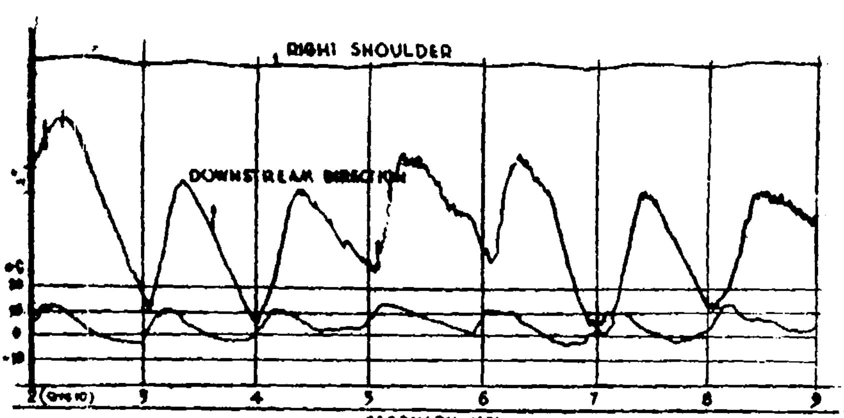

RIGHT SHOULDER
DOWNSTREAM DIRECTION
FEBRUARY 1951

Fig. 14

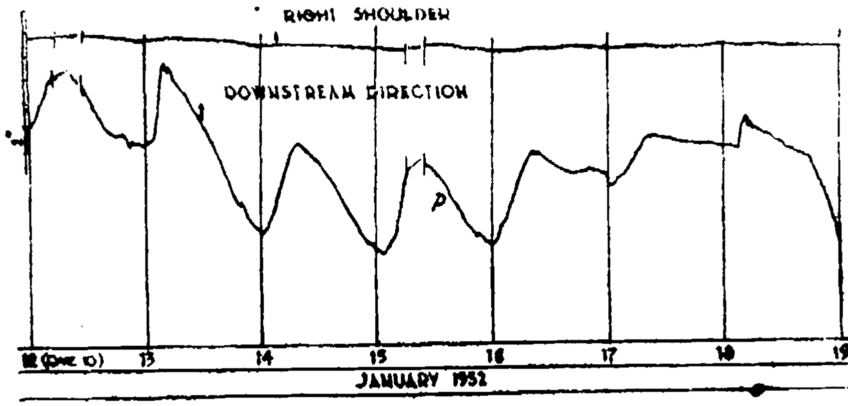



Fig. 15

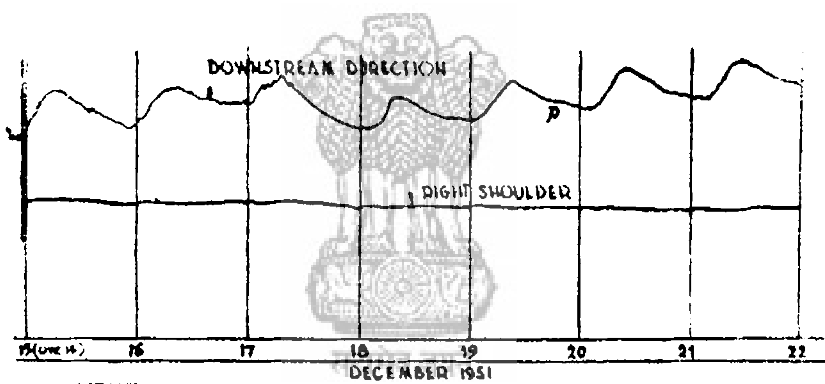



Fig. 16

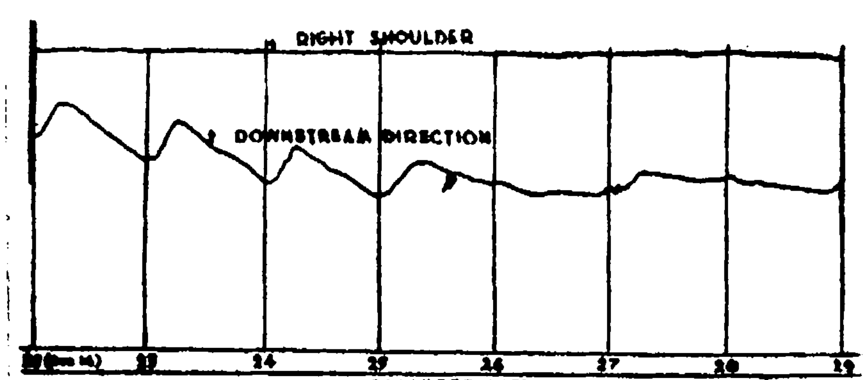



Fig. 17

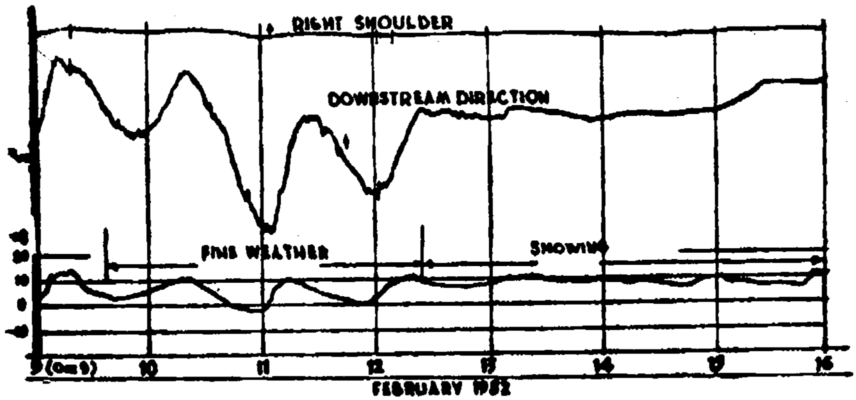



Fig. 18

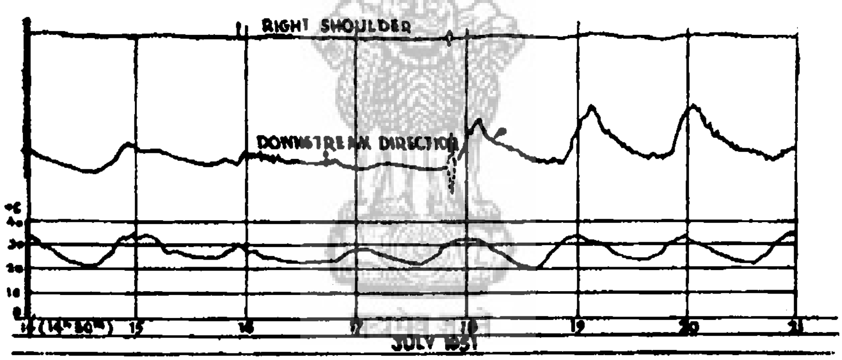



Fig. 19

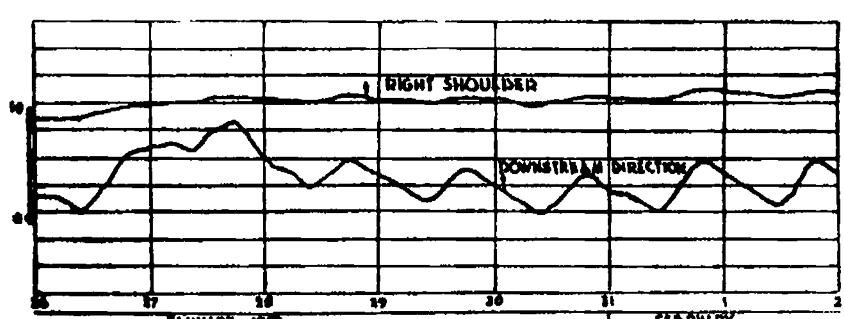



Fig. 20

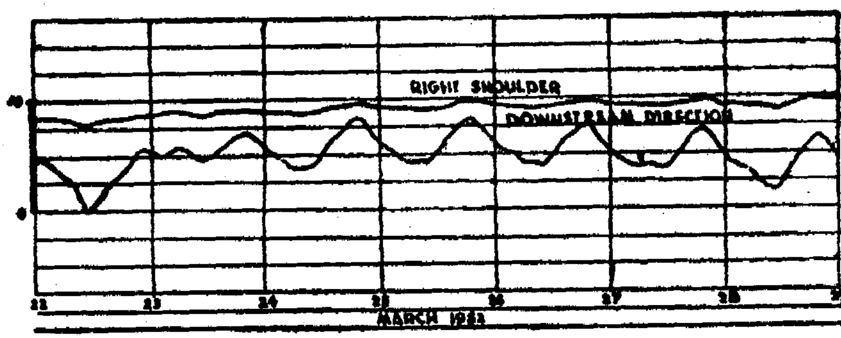



Fig. 21

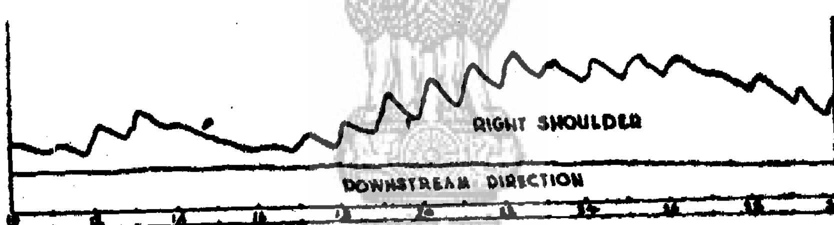



Fig 21 bis REDUCTION 3/10; IN THE ORIGINAL, 1" = 3 mm.

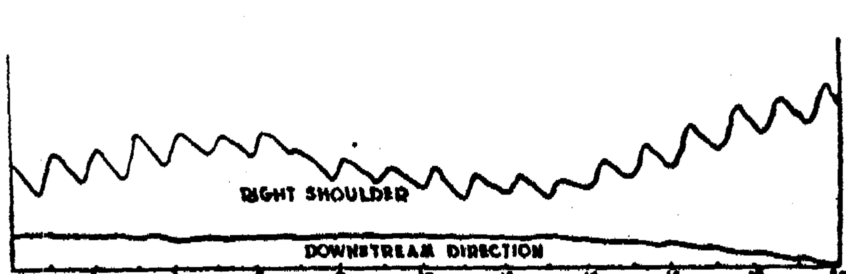



Fig. 21 ter. REDUCTION 3/10; IN THE ORIGINAL, 1" = 3 mm.

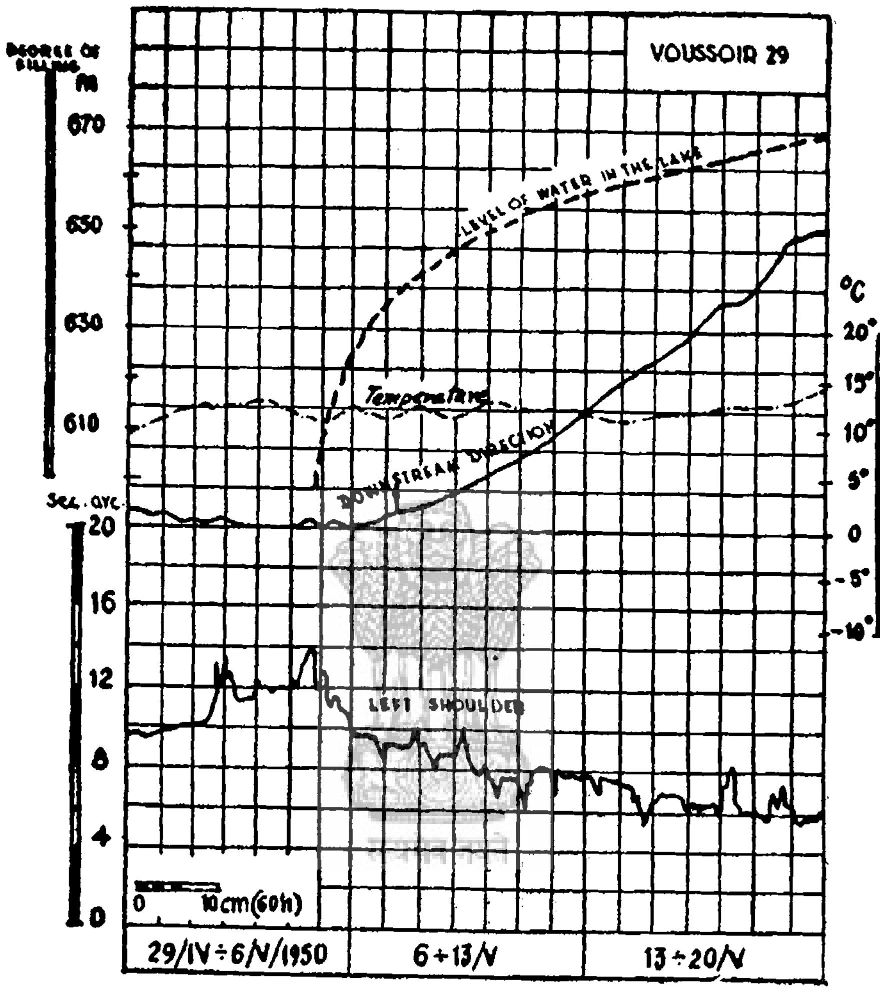

VOUSSOIR 29
DEGREE OF FILLING
m
670
650
630
610
°C
20°
15°
10°
5°
0
-5°
-10°
LEVEL OF WATER IN THE LAKE
Temperature
DOWNSTREAM DIRECTION
Sec. arc.
20
16
12
8
4
0
LEFT SHOULDER
0 10cm(60h)
29/IV÷6/V/1950
6÷13/V
13÷20/V

Fig. 22

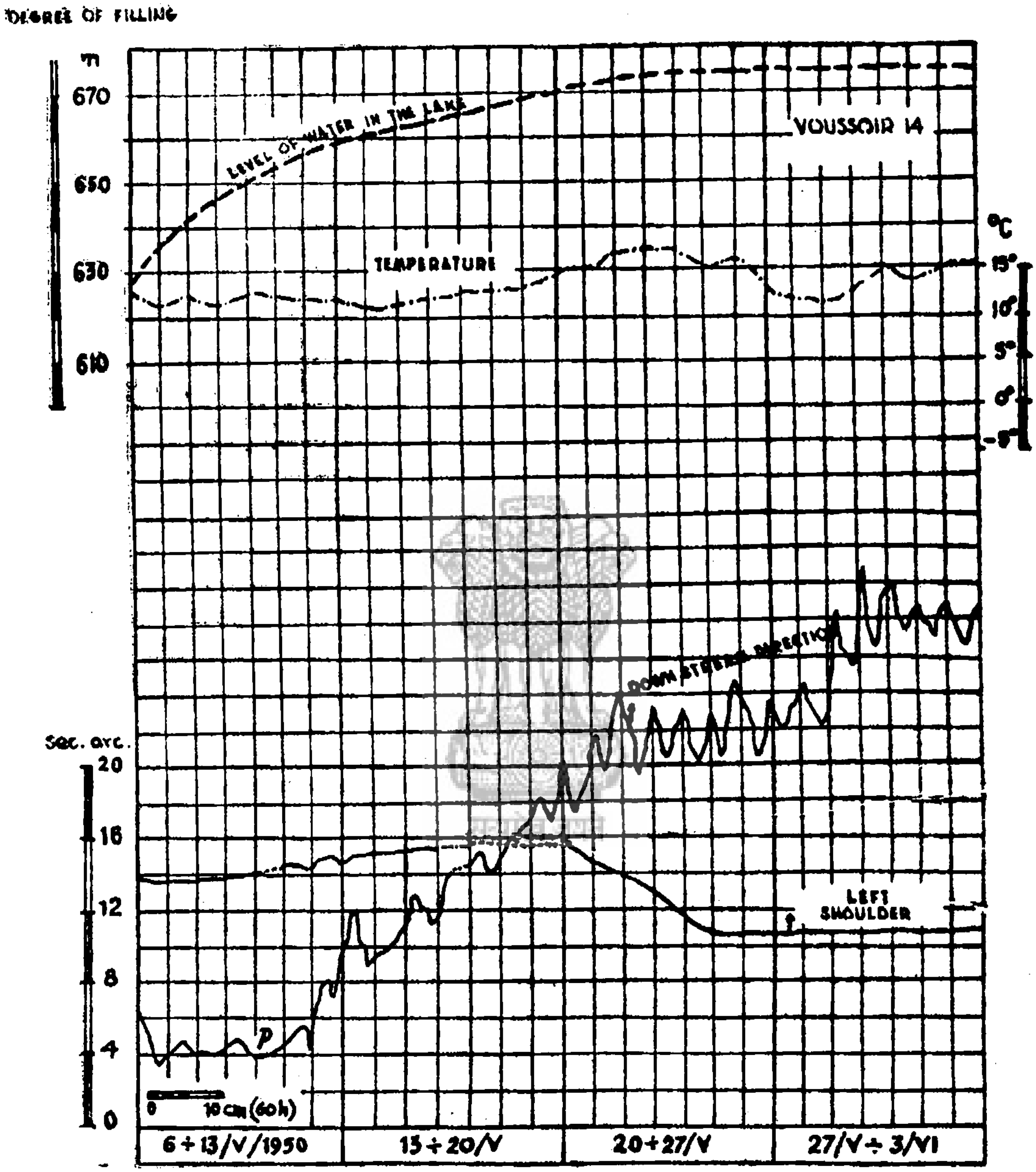

DEGREE OF FILLING
m
670
650
630
610
LEVEL OF WATER IN THE LAKE
VOUSSOIR 14
°C
TEMPERATURE
15°
10°
5°
0°
-5°
DOWN STREAM DIRECTION
Sec. arc.
20
16
12
8
4
0
LEFT SHOULDER
P
0
10 cm (60 h)
6 ÷ 13/V/1950
13 ÷ 20/V
20 ÷ 27/V
27/V ÷ 3/VI

Fig. 23

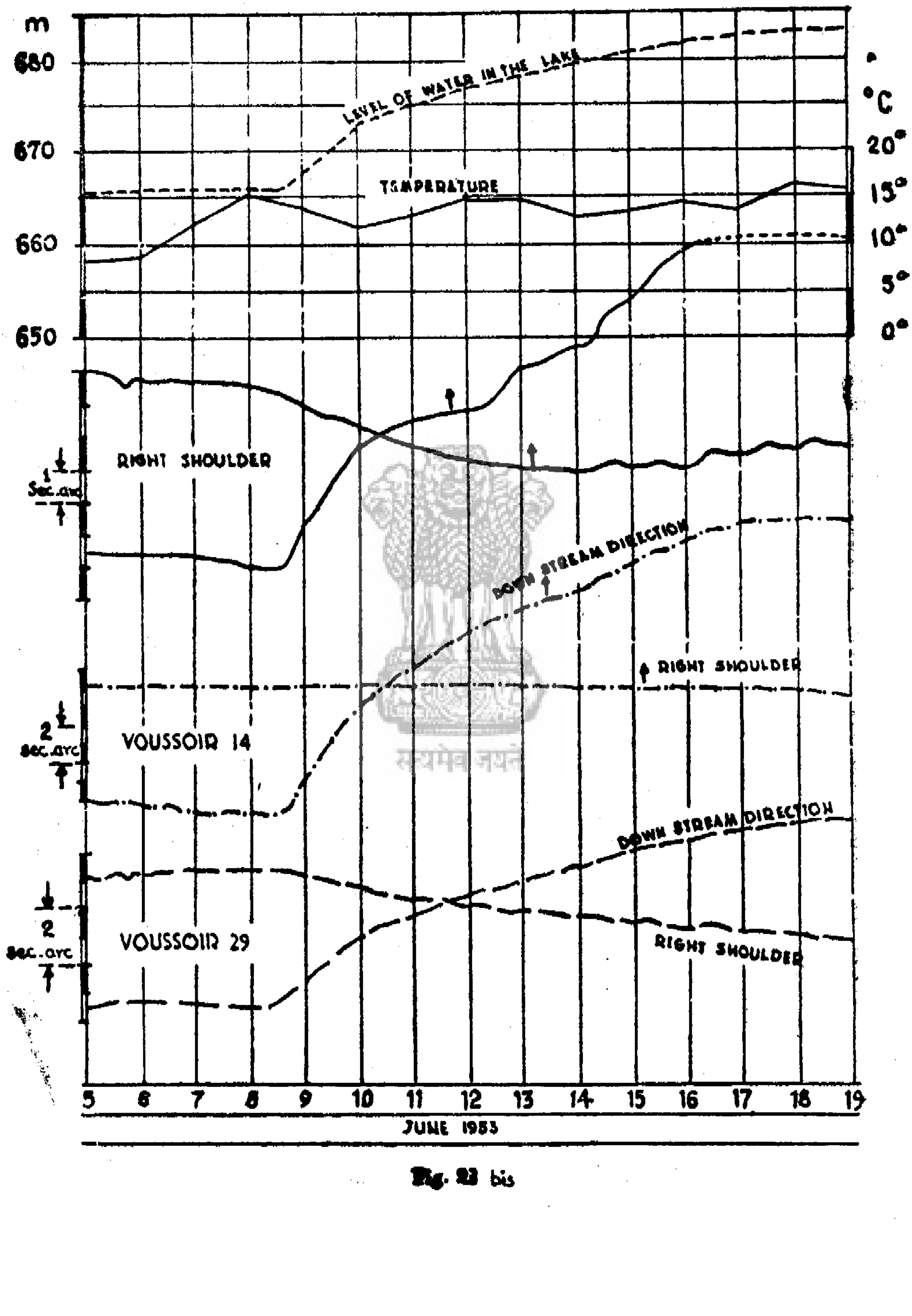

m
680
670
660
650
°C
20°
15°
10°
5°
0°
LEVEL OF WATER IN THE LAKE
TEMPERATURE
RIGHT SHOULDER
1 Sec.arc
DOWN STREAM DIRECTION
RIGHT SHOULDER
2 sec.arc
VOUSSOIR 14
DOWN STREAM DIRECTION
2 sec.arc
VOUSSOIR 29
RIGHT SHOULDER
5 6 7 8 9 10 11 12 13 14 15 16 17 18 19
JUNE 1953

Fig. 23 bis

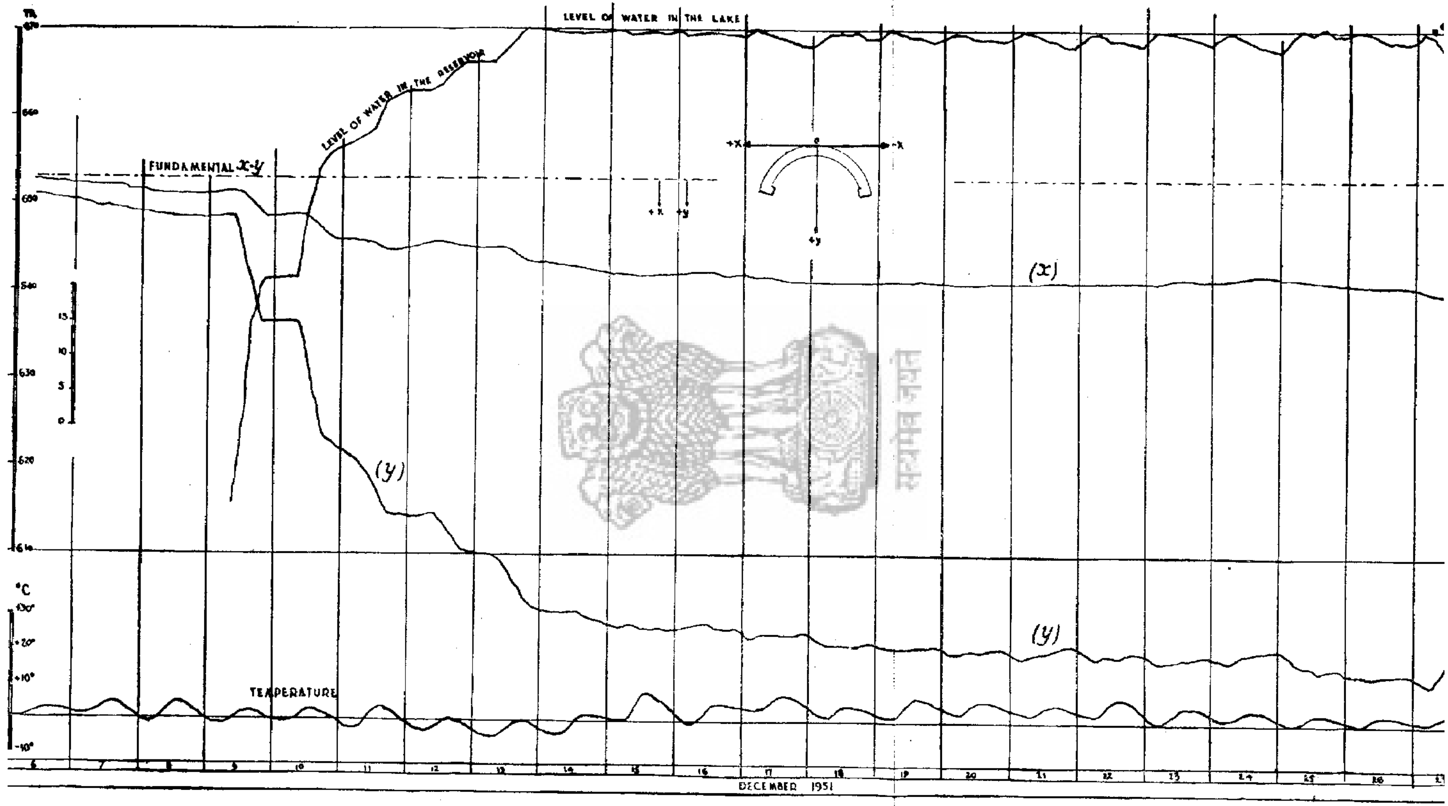



Fig. 24

N&PC/63—8

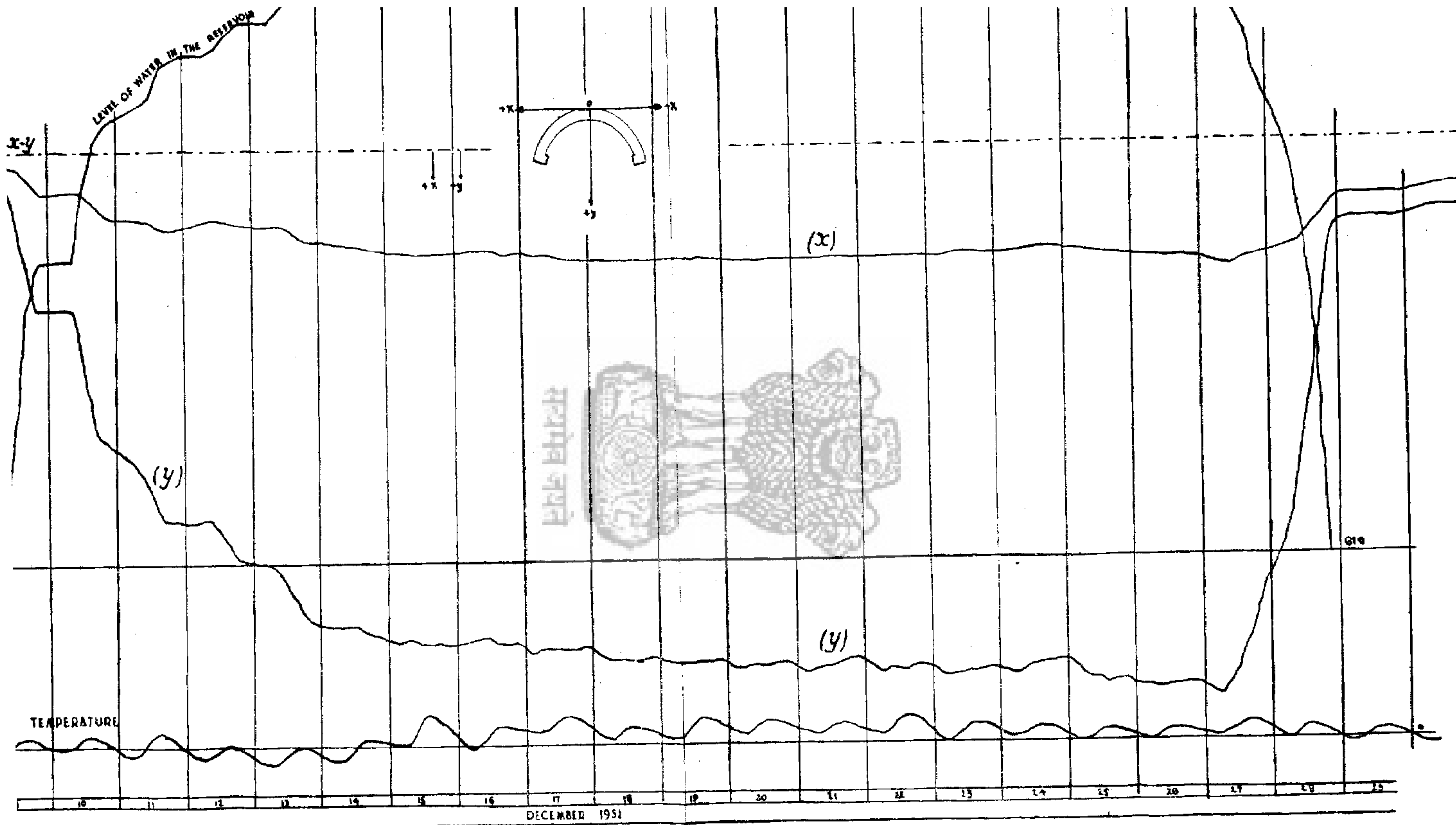
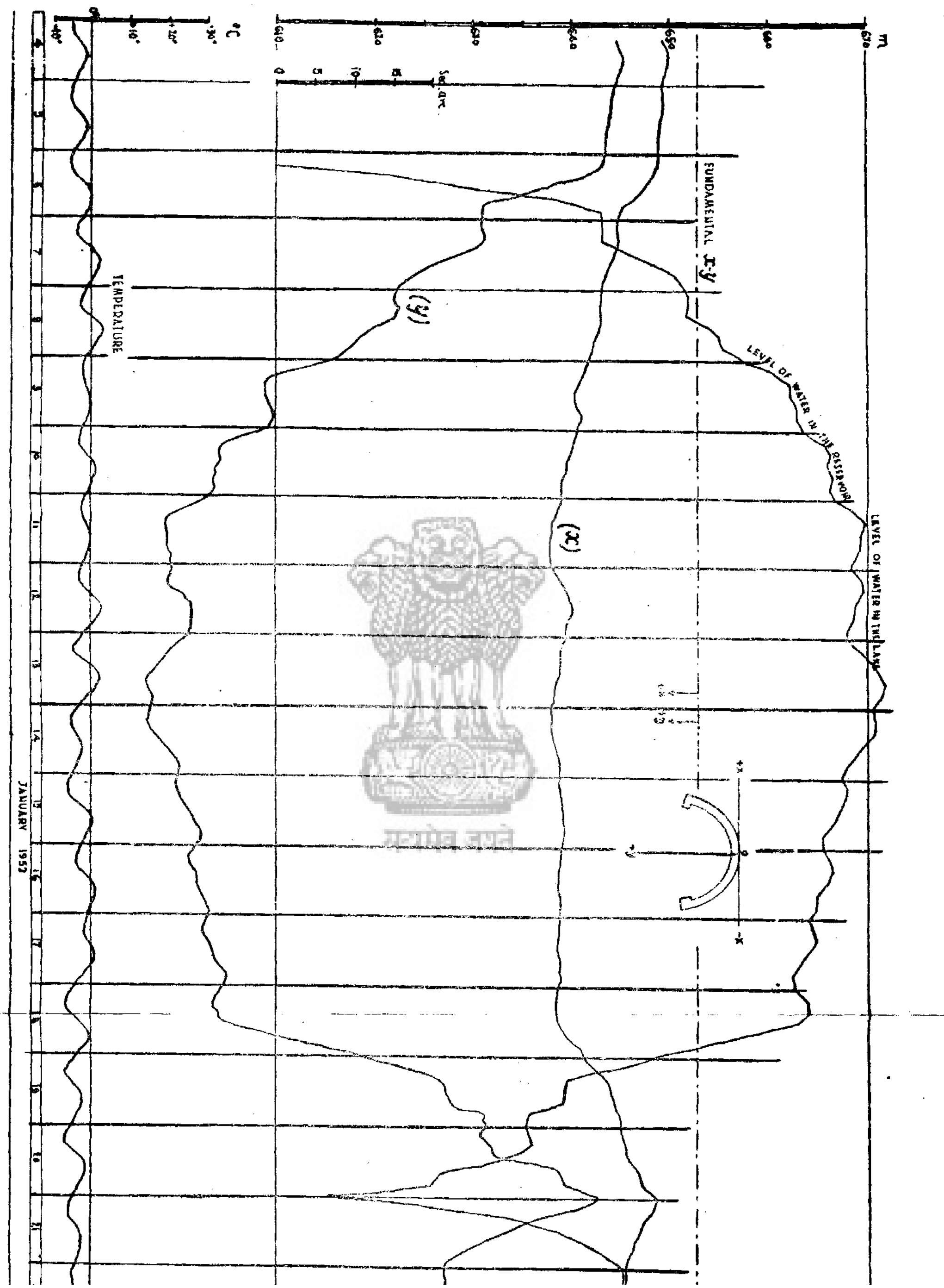

LEVEL OF WATER IN THE RESERVOIR
x-y
+x
+y
(x)
(y)
(y)
G19
TEMPERATURE
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
DECEMBER 1951

Fig. 34



m
610
620
630
640
650
660
670
°C
TEMPERATURE
FUNDAMENTAL x-y
LEVEL OF WATER IN THE RESERVOIR
LEVEL OF WATER IN THE LAKE
(x)
(y)
JANUARY 1952


Fig. 23

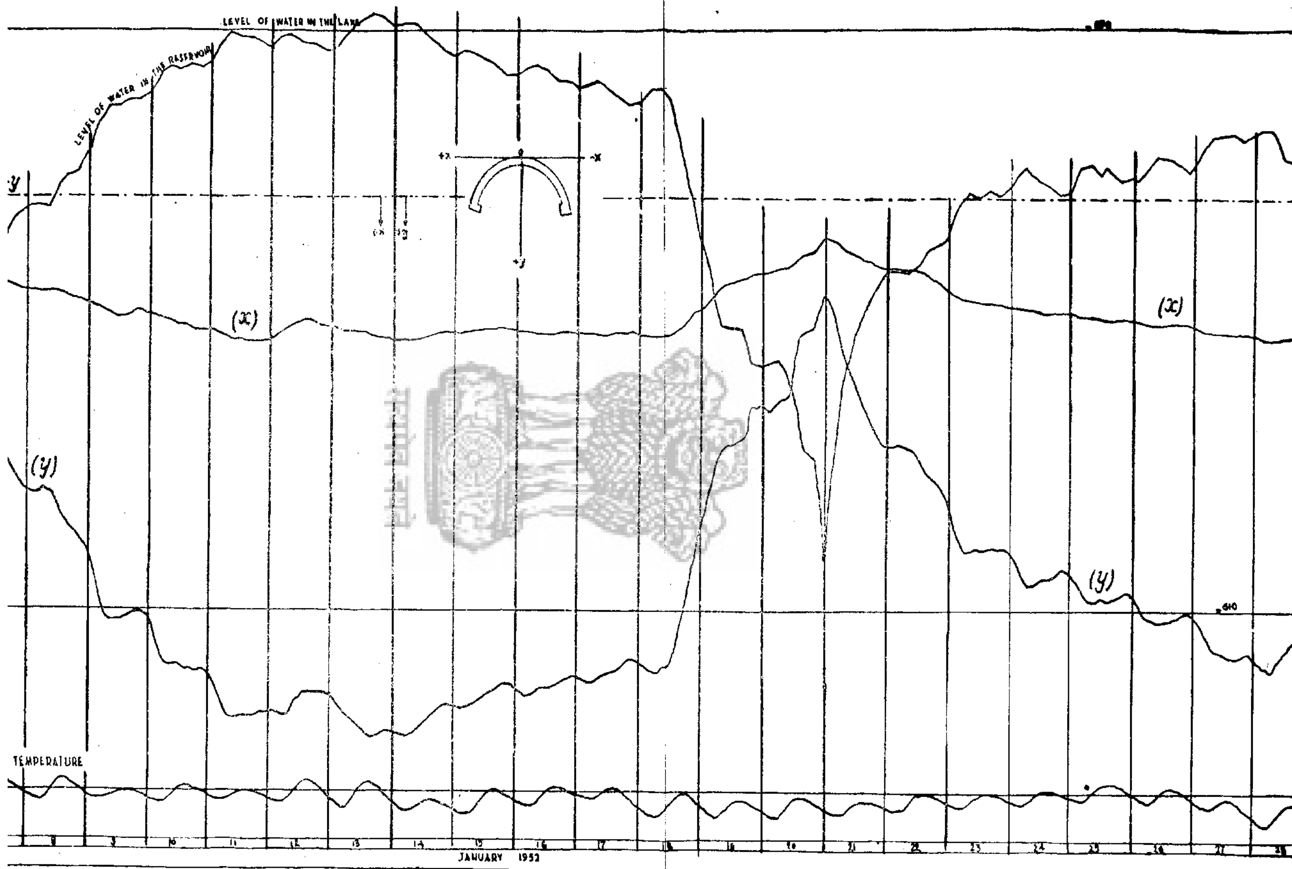

LEVEL OF WATER IN THE LAKE
LEVEL OF WATER IN THE RASERVOIR
(x)
(y)
(x)
(y)
+x
-x
+y
TEMPERATURE
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
JANUARY 1952

Fig. 25

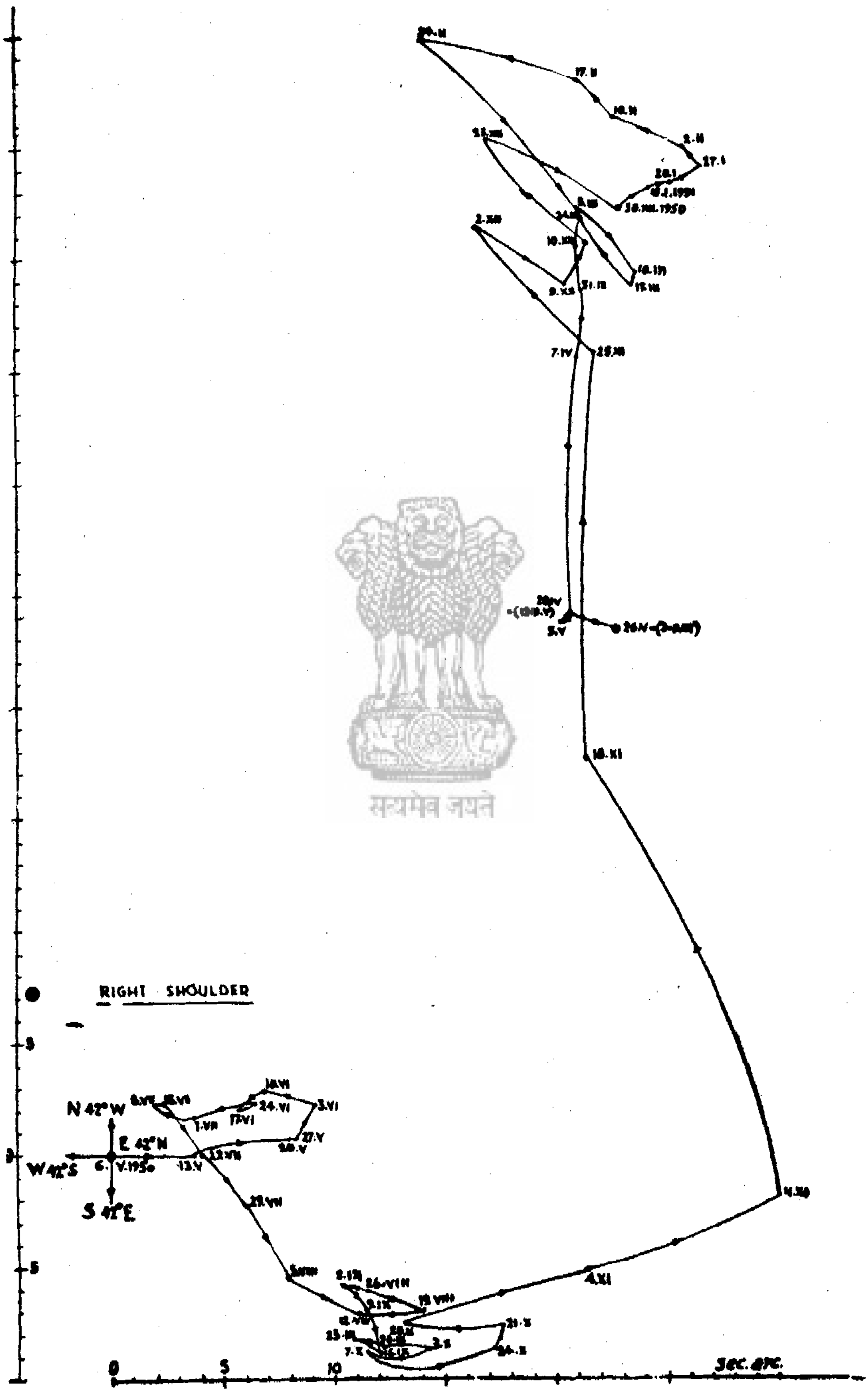



Fig. 26

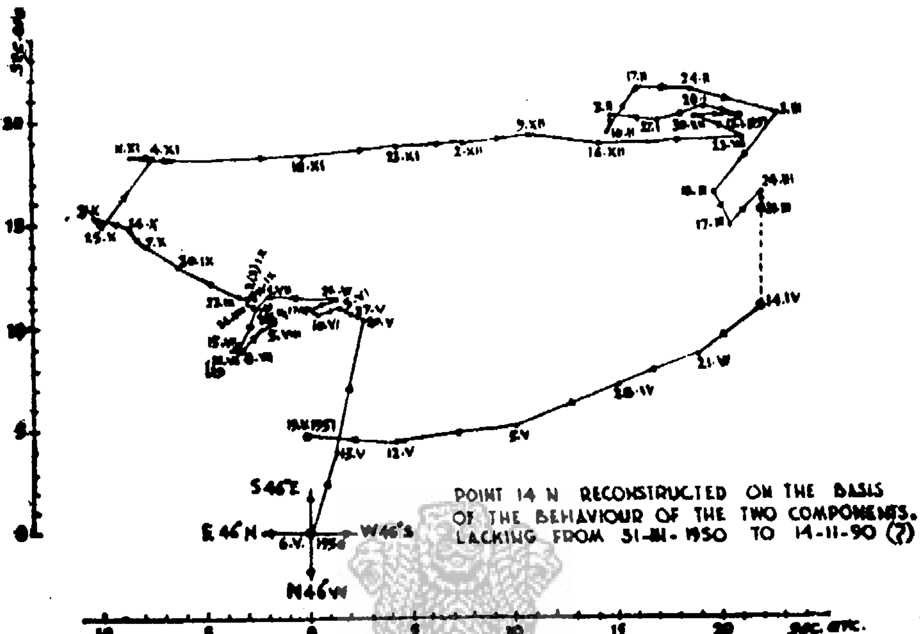



Fig. 27

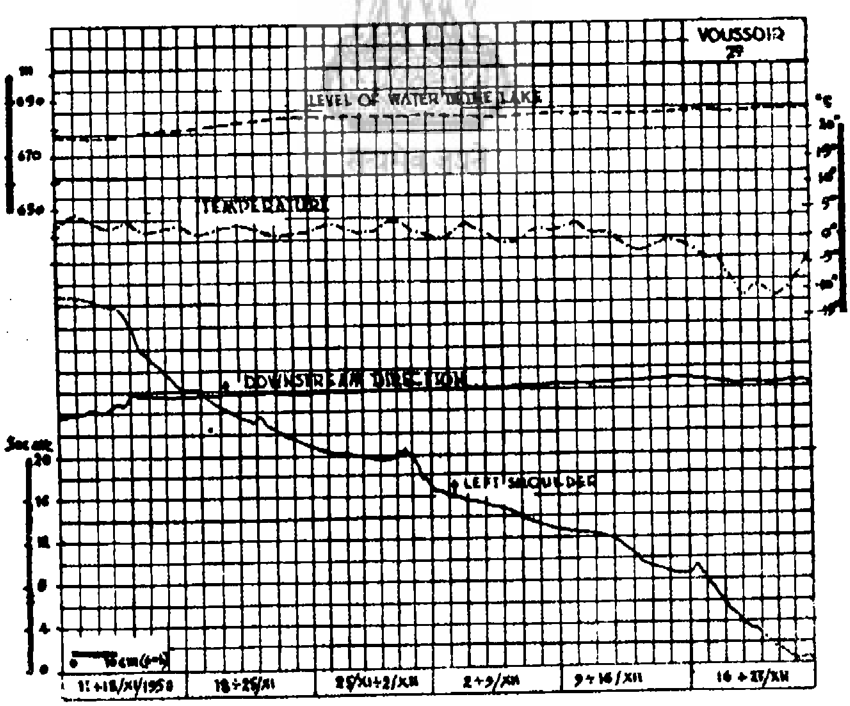



Fig. 27 bis

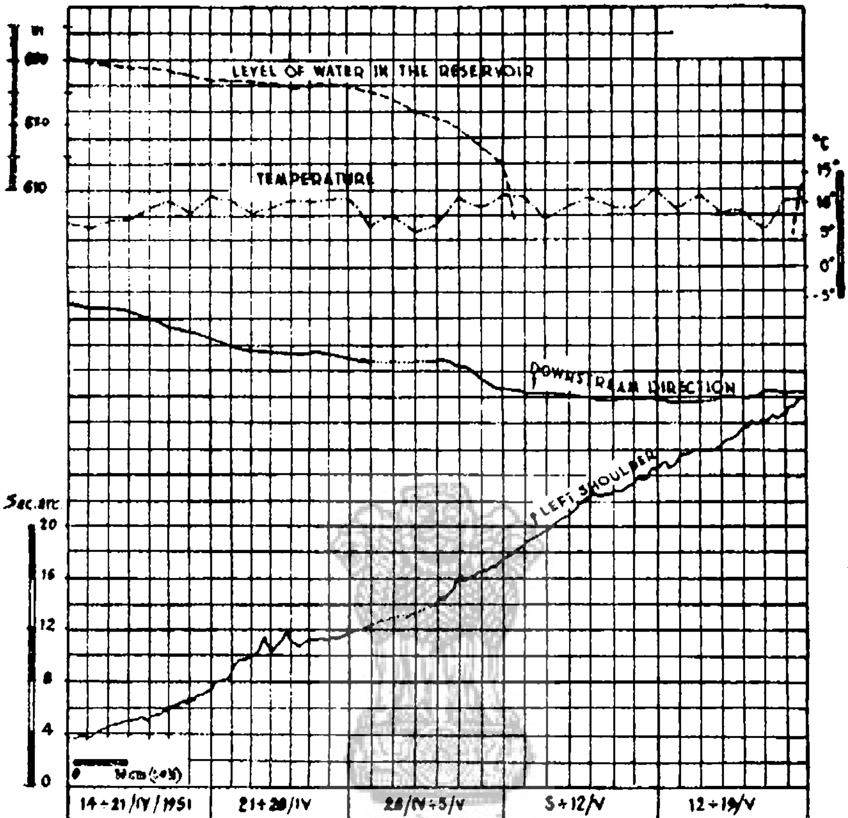

LEVEL OF WATER IN THE RESERVOIR
TEMPERATURE
DOWNSTREAM DIRECTION
LEFT SHOULDER
°C
15°
10°
5°
0°
-5°
Sec. arc
20
16
12
8
4
0
610
14÷21/IV/1951
21÷28/IV
28/IV÷5/V
5÷12/V
12÷19/V

Fig. 28

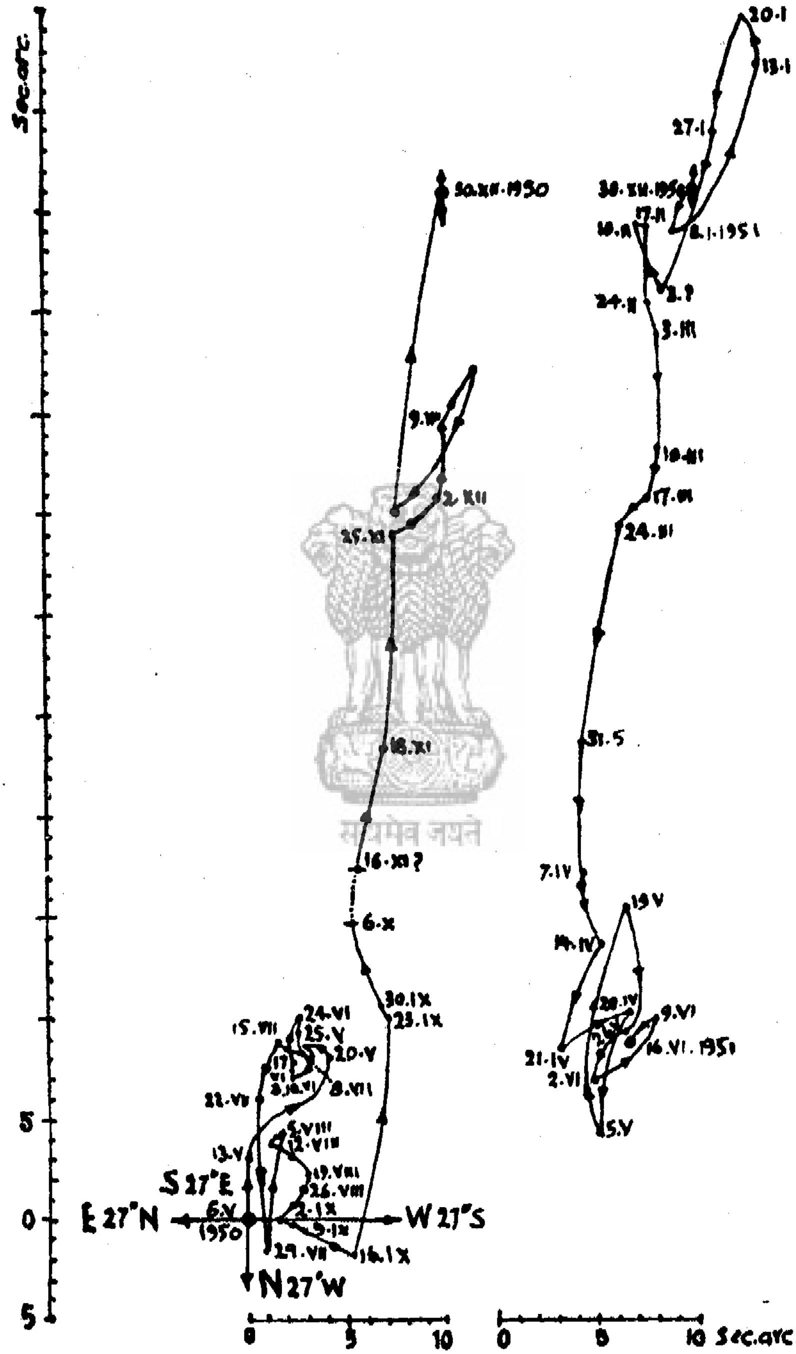

Sec. arc
30.XII.1950
20.I
13.I
27.I
30.XII.1950
8.I.1951
31.5
19.V
16.VI.1951
5.V
S 27°E
E 27°N
W 27°S
N 27°W
6.V
1950
5
0
5
0
5
10
0
5
10 Sec. arc.

Fig. 29

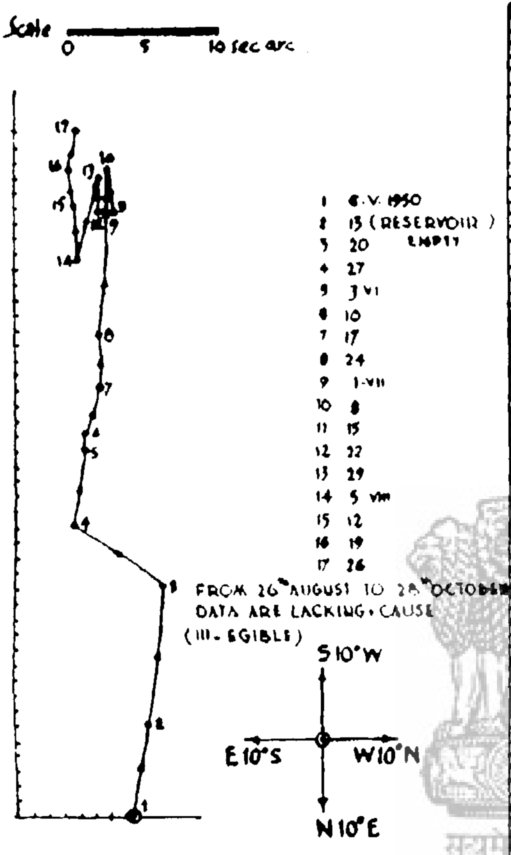



Fig. 30

FIG. 30 GIVES THE MOVEMENTS OF VOUSSOIR XIV FROM THE COMMENCEMENT OF FILLING UP TO THE POINT OF FULL CHARGE, SUPERPOSED ON THE SEASONAL EFFECT OF SUMMER (THRUST TOWARDS THE DOWNSTREAM DIRECTION).

FIG 30 bis. GIVES, AFTER A STATIONARY INTERVAL, THE WINTER SEASONAL EFFECT SUPERPOSED ON THE EFFECT OF GRADUAL DISCHARGE OF RESERVOIR AND OTHER SEMI-PERIODIC CAUSES OF DISTURBANCES.

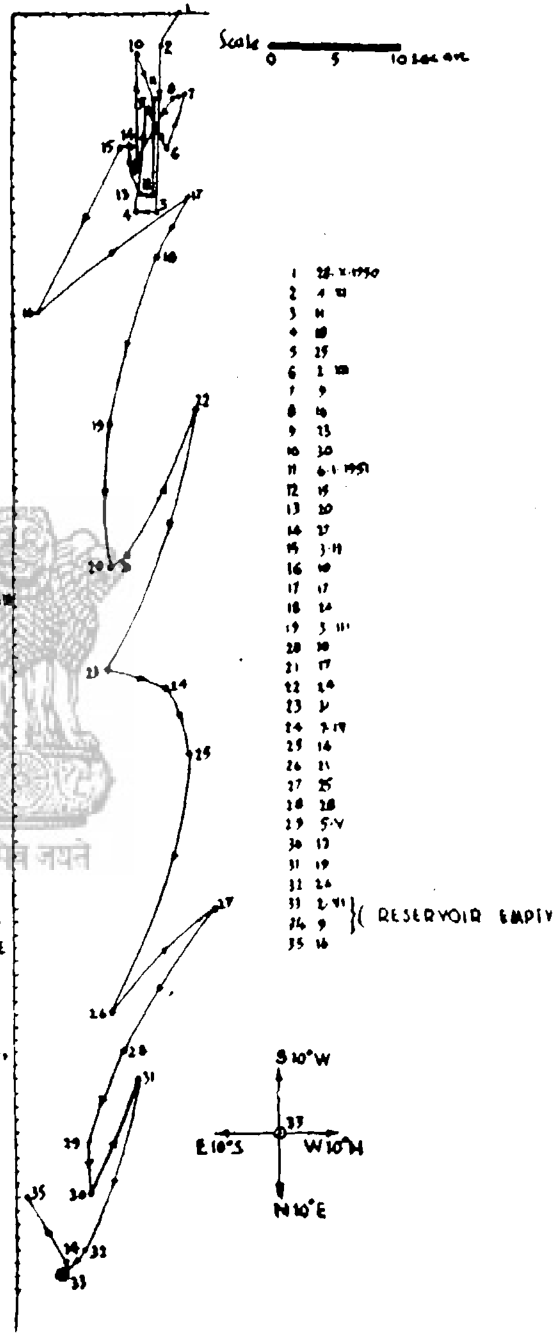



Fig. 30 bis

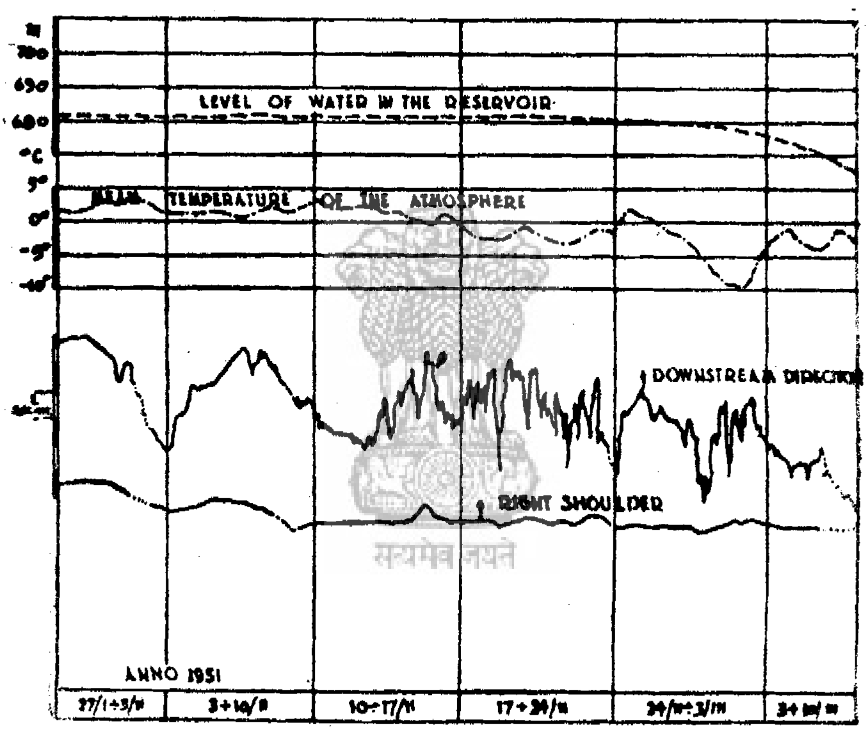

LEVEL OF WATER IN THE RESERVOIR
MEAN TEMPERATURE OF THE ATMOSPHERE
DOWNSTREAM DIRECTION
RIGHT SHOULDER
ANNO 1951

Fig. 31

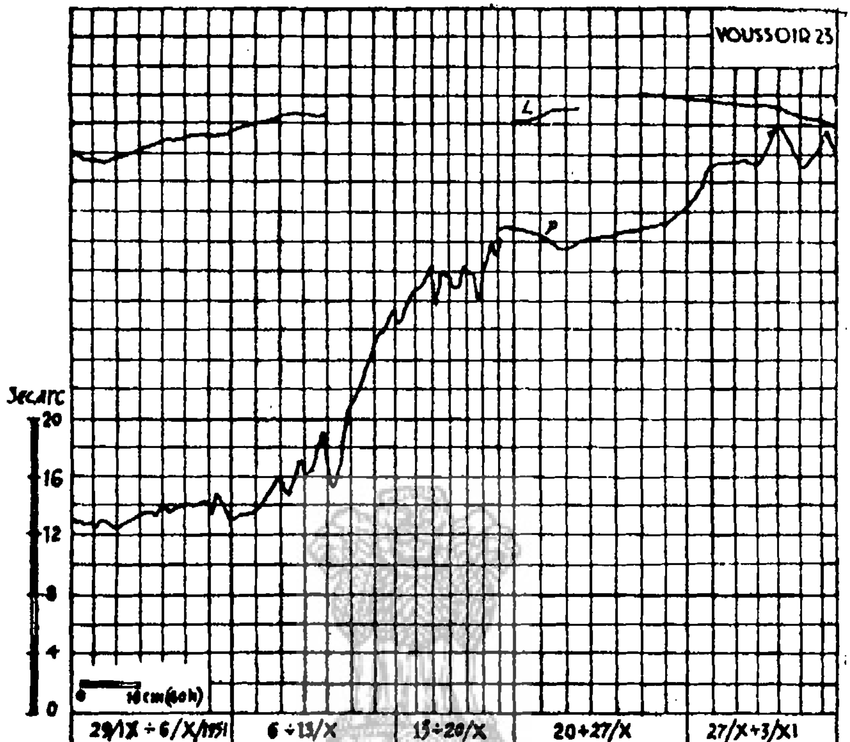



Fig. 32

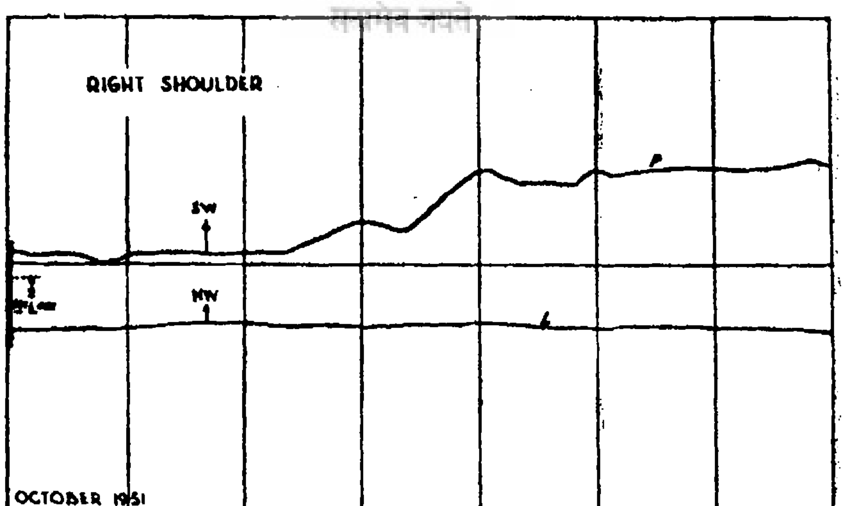



Fig. 32 bis

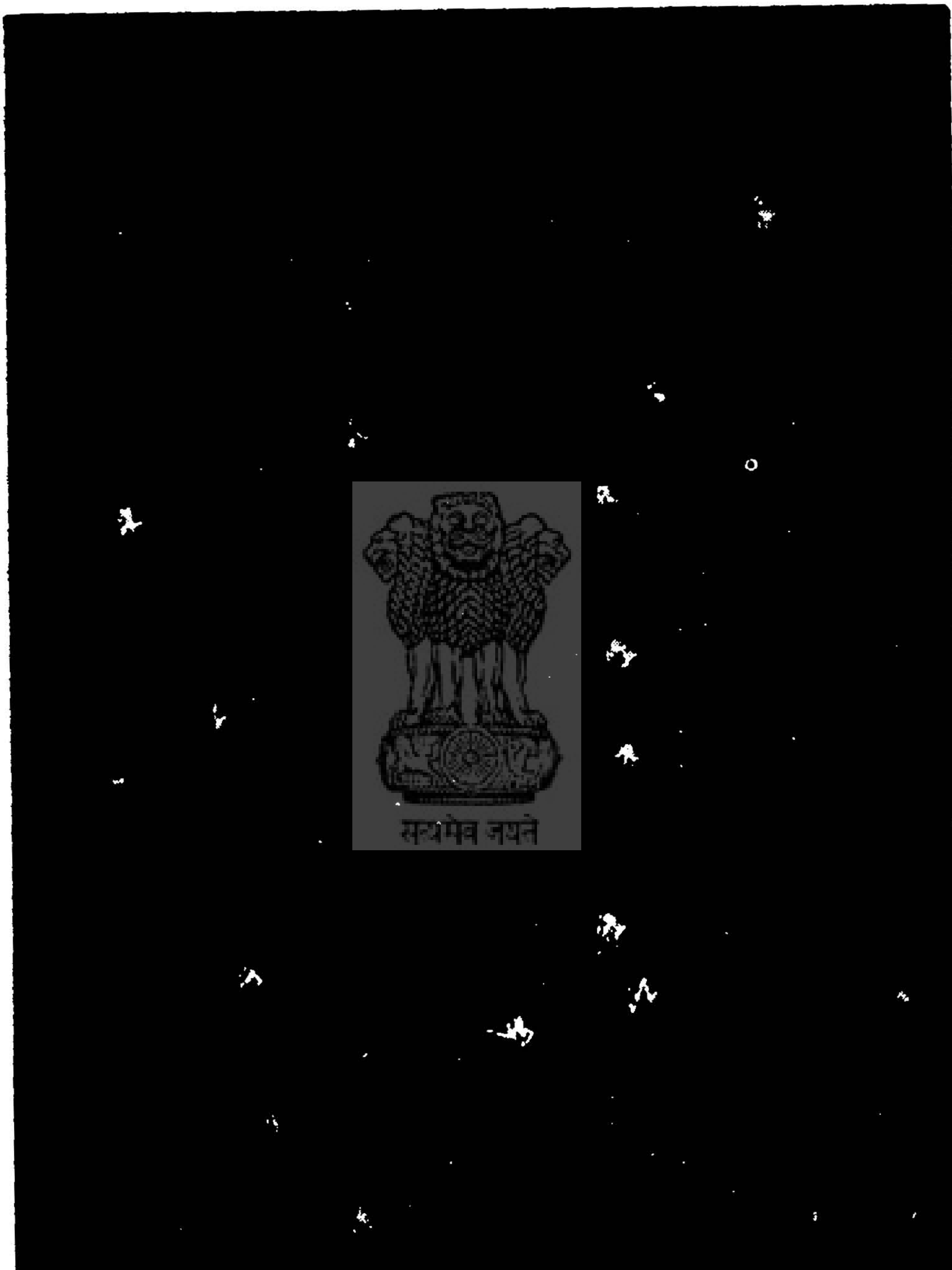




Fig. 33

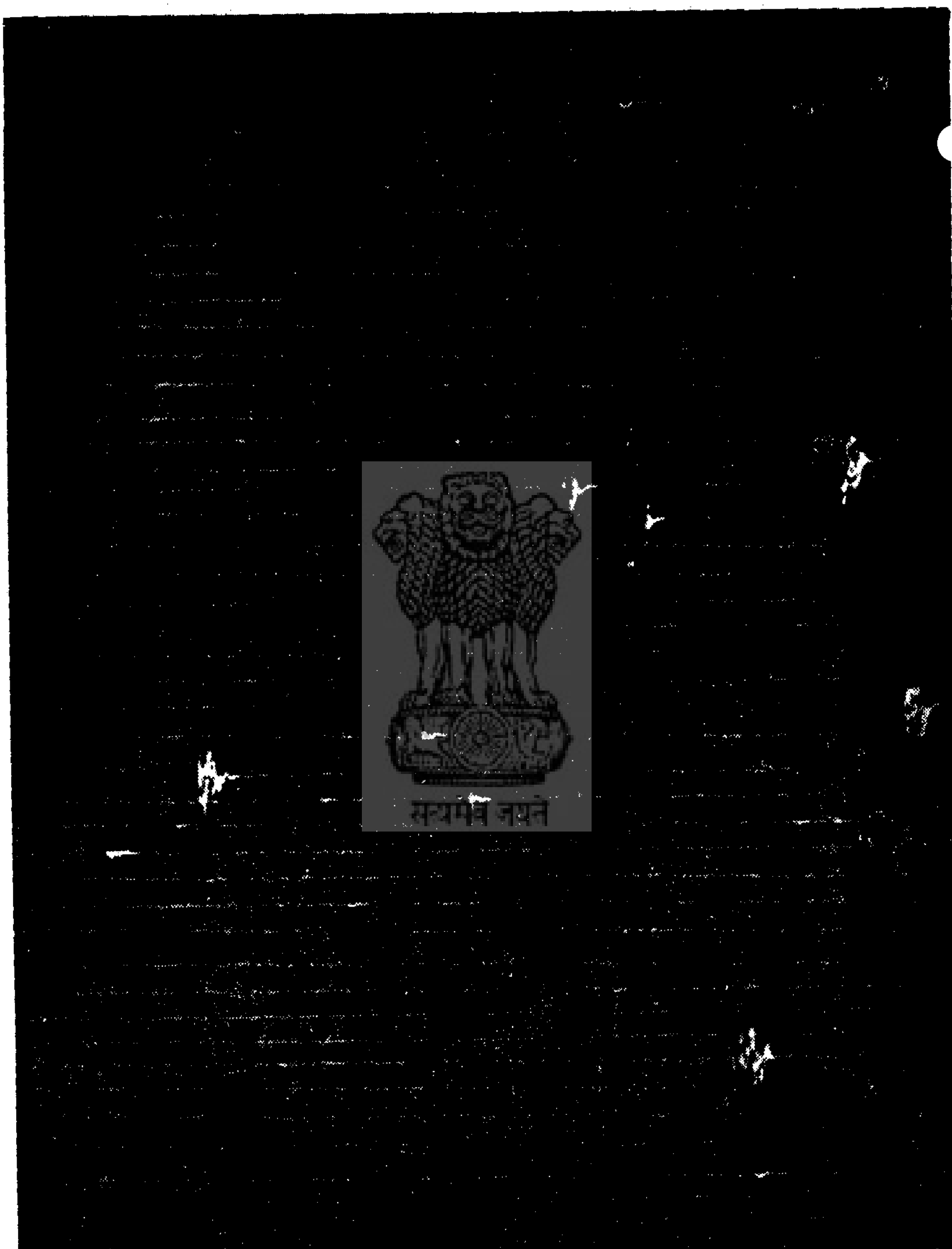



Fig. 34

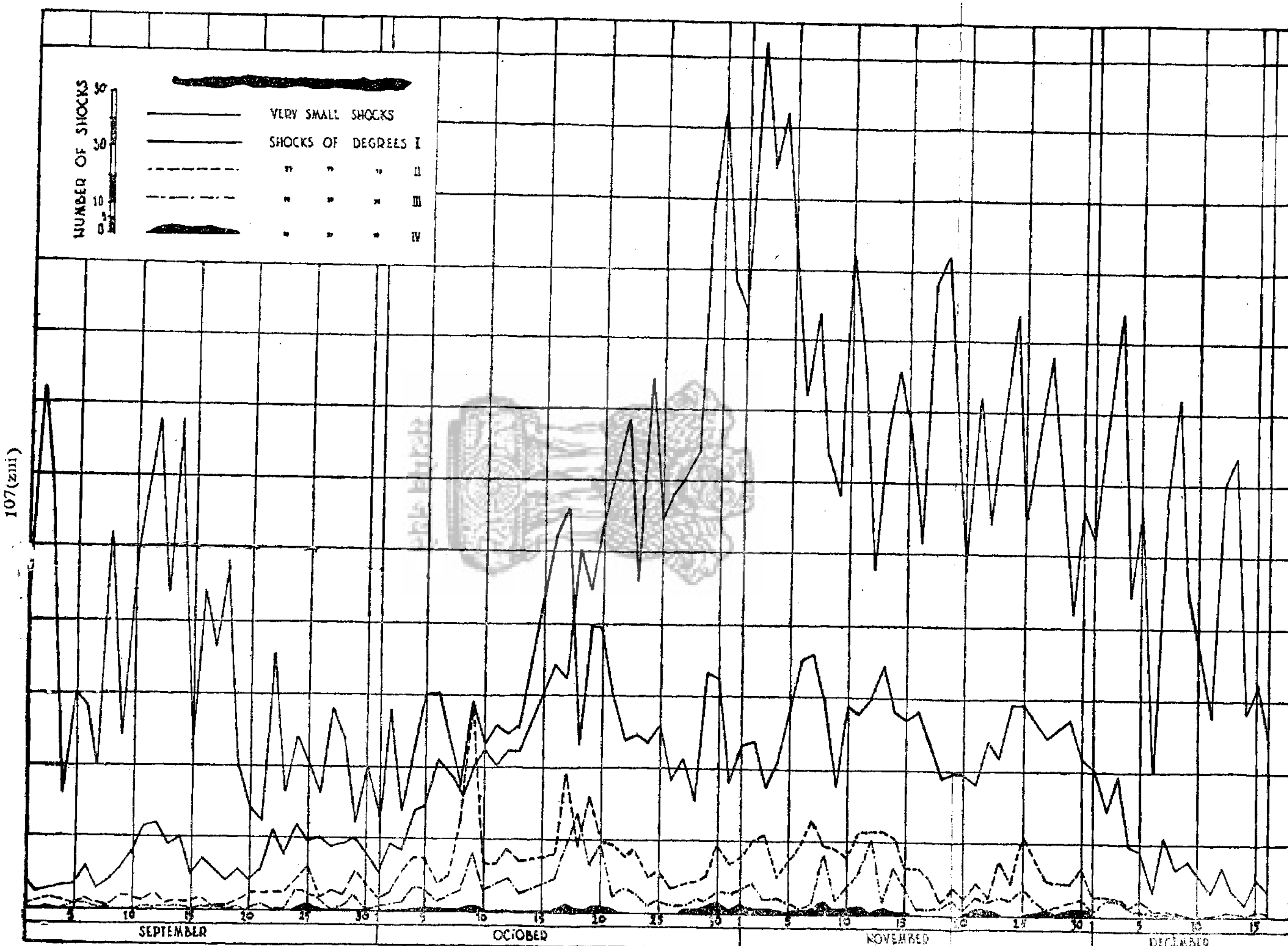

NUMBER OF SHOCKS
50
30
10
5
0
VERY SMALL SHOCKS
SHOCKS OF DEGREES I
" " " II
" " " III
" " " IV
5
10
15
20
25
30
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER

Fig. 33

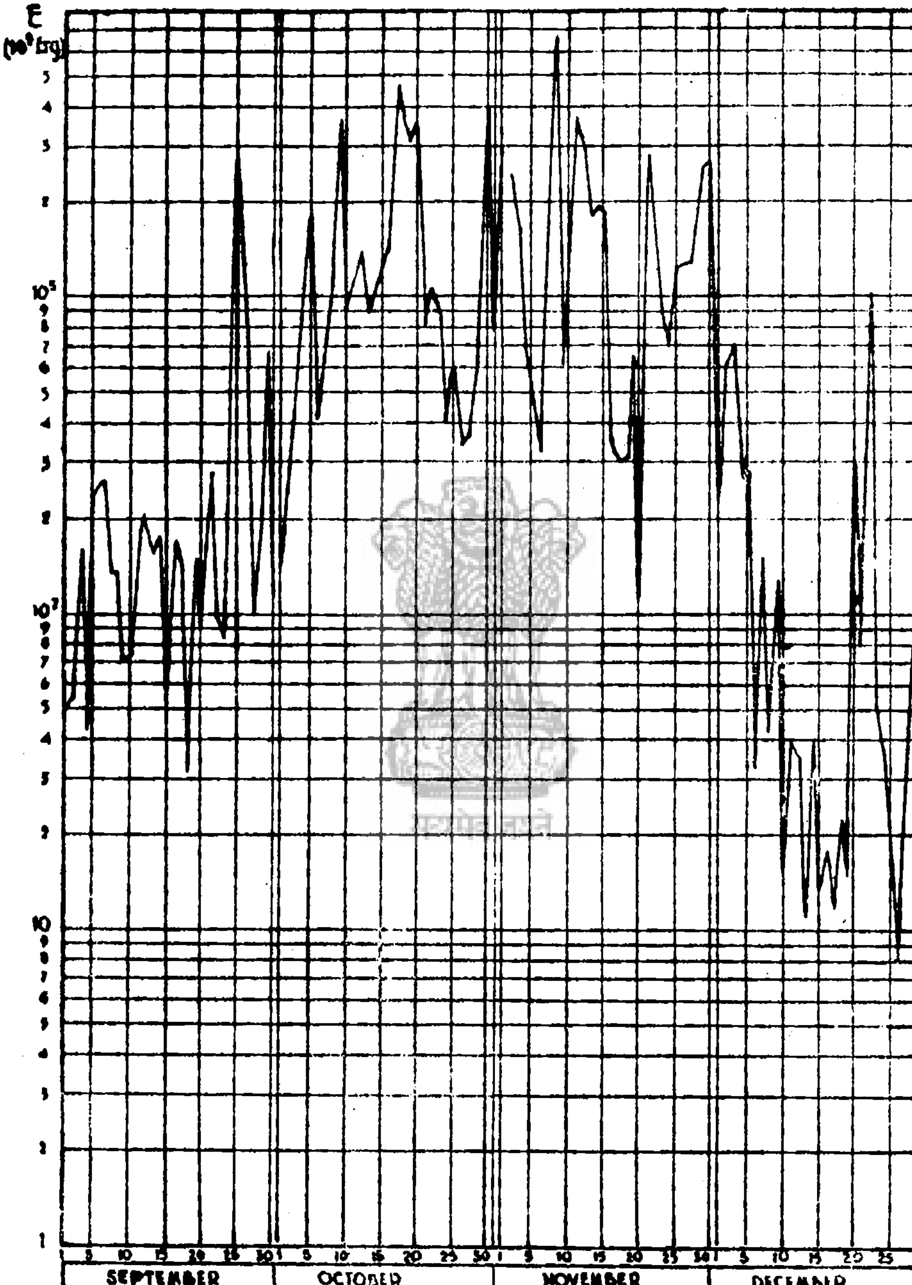



Fig. 36

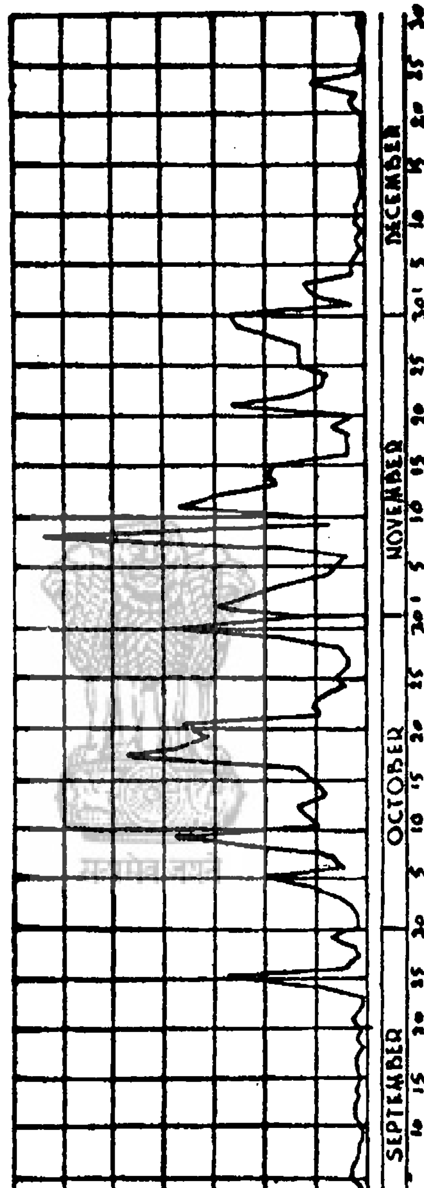



Fig. 37 (ALONG THE ORDINATES, 1 UNIT EQUALS $10^{12}$ ERG)

Fig. 42—Rapid Oscillations of period 1/10 sec, approx.

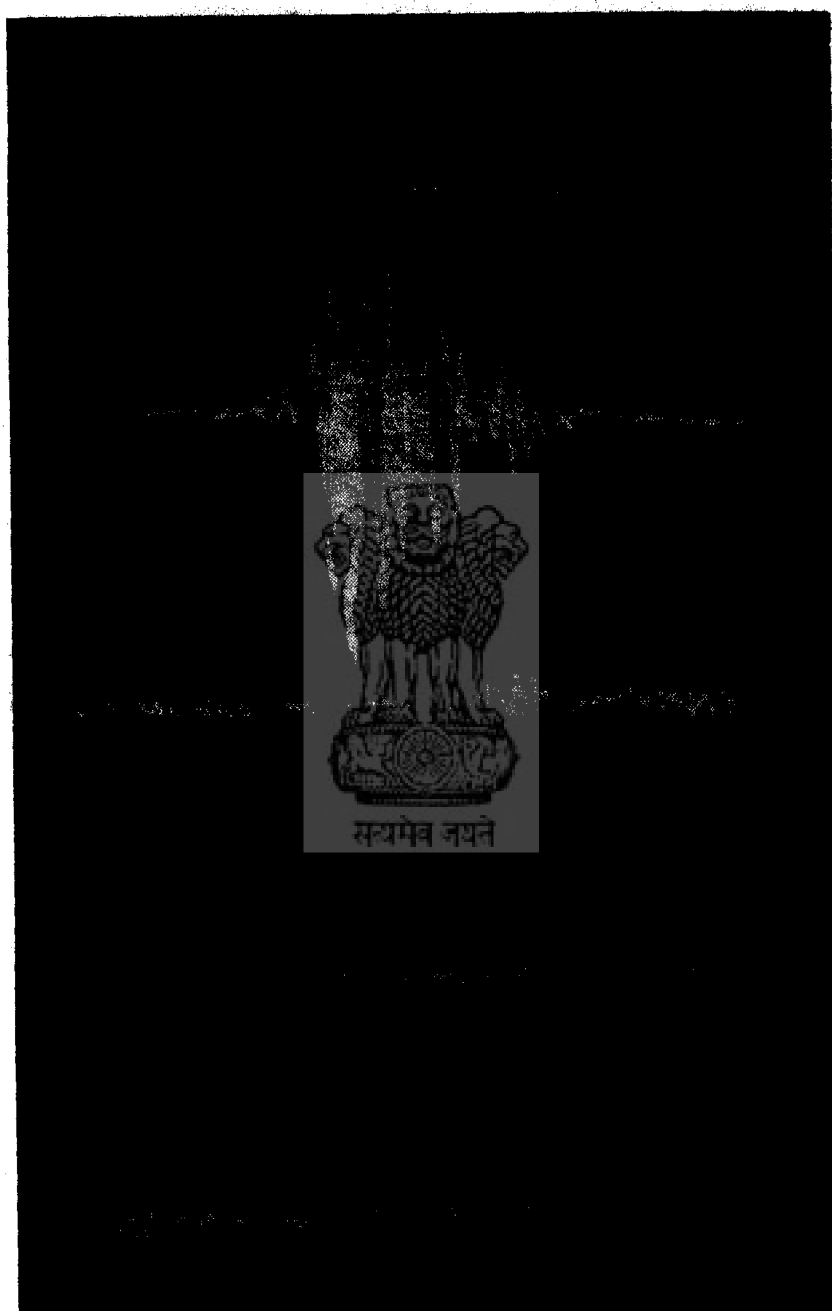




**Fig. 43—Microseismic vibrations with period of 1/10 sec. approx.; an example of minute shocks**

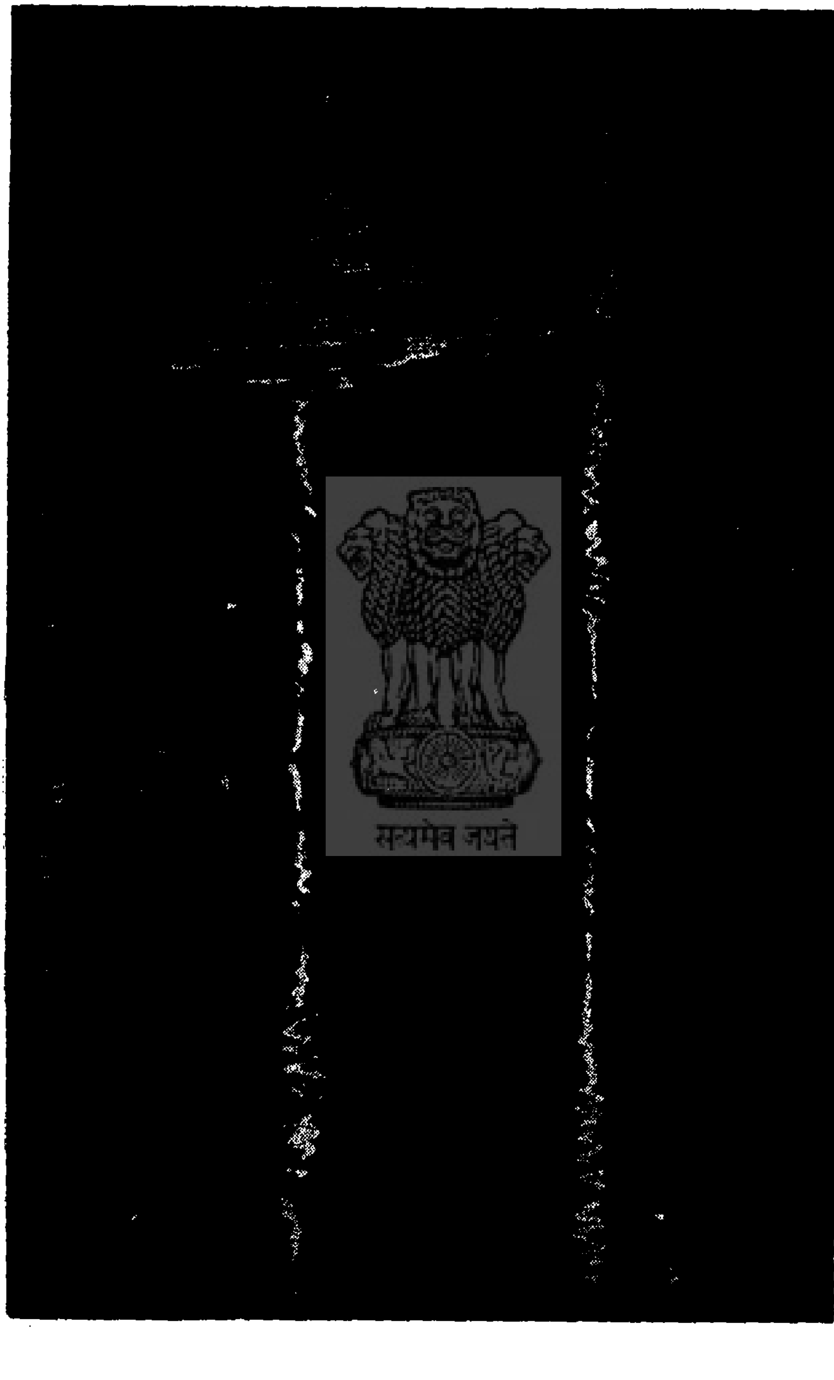



Fig. 44

## MEASUREMENTS FOR THE SEISMIC CHECKING OF DAMS

(1) The structural importance of modern dams and the possibility of their construction even in highly seismic zones have led in recent years to the development of a special technique for the checking of dams with respect to possible seismic events. On this account we think it is opportune to give in this paper some references to what has been done in respect of its dams by the Adriatic Society for Electricity (Societa Adriatica di Electricita, S.A.D.E.), especially because the society has for its consultant an expert of the eminence of Prof. Pietro Caloi and the warm cooperation of the National Institute of Geophysics.

Our general practice is to establish, in the neighbourhood of the dam under examination, obviously when the dam has a certain importance or the seismicity of the zone makes it advisable, a complete seismic station with all the three components as well as a certain number of clinographs distributed over the foundation rock and over the structure itself. In some cases observations are limited to those of the clinographs.

Of the seismic stations serving our dams, two are already functioning and a third is in course of construction.

Recalling that the dams of the S.A.D.E. cover an area of about 3,000 km$^2$, we have a mean density of one station for about 1,000 km$^2$. This allows us to obtain, besides a knowledge of the total irregularities, a very good idea of the seismic character of the region in which we are concerned, and so of the hebaviour of such of our dams that do not have their own seismic station.

The first seismic station to be installed was that attached to the dam, Pieve di Cadore. It is situated in the control cabin and is thus almost exactly over the left foundation of the dam. The station is provided with three seismographs, two for the horizontal components and one for the vertical, as well as a radio received for the reception of time signals, a regulating chronometer with accessories, and the seismic observations are supplemented by those of 5 Zollner-Caloi* clinographs distributed over the foundation rock and the structure itself. This seismic station, which forms a part of the geoseismic network of the National Institute of Geophysics commenced observation in 1949 and has been functioning regularly from that year.

The second station is that of Tolmezzo situated in the Services Office of the S.A.D.E., which is 3 km distant from the zone where the Ambiesta Dam rises. This station has Girlanda three-component seismographic instruments. It began to function in 1951 in the Salesian Institute of Tolmezzo and was transferred to the place mentioned

*Pietro Caloi-Emma De Rossi. Diamanti—"The Photoclinograph with horizonta pendulum; theory and applications".

in 1955, where it has been functioning regularly from that year. In the neighbourhood of the dam and over the structure itself are installed 7 Zollner-Caloi clinographs.

The third station is under construction at the left abutment (command cabin) of the Vaiont Dam. The construction will be completed in 1960. The three-component Girlanda seismographs are, however, functioning already since 1959. Like the preceding, this station also is supplemented by 2 Zollner-Caloi clinographs placed over the foundation rock and 5 more are being installed on the structure itself.

Other clinographs have been installed at the Val Gallina Dam (one) and the Pontesei Dam (three).

In total, we have three seismographic stations and three clinograph stations. In addition, measurements of rotation are made in the dams mentioned and in those of Lumiei, Fedaia and Barois with direct reading clinometers and pendulums, the observations being generally taken twice a week. These readings give us the amount of rotation which has occurred in the interval between two successive readings, while the clinographs (which are also very much more sensitive) provide a continuous recording of the rotations.

It should be observed that the seismic and clinograph stations are only a small part of the organisation for keeping a check on the dams, of which the physical characteristics, the deformations and the displacements are under continual observation. For example, we may recall that for the Pieve di Cadore Dam we have about 800 measurement points in addition to a topographical survey network of great precision with 60 bench marks. The Vaiont Dam will have a network of measuring stations of about the same magnitude.

(2) The seismic measurements for the checking of dams, in addition to their general interest, are of special importance particularly in the initial phase of the mutual adjustment between the dam and its foundations. This active phase lasts over a considerable period of time. The heavy hydrostatic loads caused by the accumulation of the water behind the dam, in fact, do disturb a geomorphological equilibrium established over a long period of time, and this originates a succession of "adjustment" microseisms. These are at most observable instrumentally and they are too small to have any effect on the structure. In fact, it is of great importance to the persons who construct a dam and those who are in administrative charge of it to know if the displacements of the structure which are observed are due to changes in the hydraulic load, or are due to thermal effects etc. or if they orginate from the adjustment of the structure itself to a new state of equilibrium.

In this connection the investigations made about the Pieve di Cadore Dam by Caloi* are of great interest. They show, among other things, how, following on the construction of the dam, the rock systems of the two river banks, which had different elastic characteristics, but geological ages, acquired a relative dynamic independence. This "independence" prevents the accumulation of tensions

*Pietro Caloi—"Seismical and clinographic observations in the neighbourhood of large barrage dams"—Ann Geofis VI (1953) No. 3.

of a certain magnitude, by the formation of small but numerous seisms. The latter are probably due to the accumulation of a small amount of energy and its discharge by wave formation along small fracture planes, probably existing in the transition zone between the two systems. In the case of the Pieve di Cadore it seems that we cannot speak of two different geodetic blocks for the two banks, since such blocks are subject, as is well known, to movements of ascent and descent, but without any appreciable deformation. It is of special importance to be able to determine the existence of such blocks at the time of designing the dam. Clinographic observations in the region under examination (it is usual to erect clinograph stations on both the banks of the river to be dammed), continued for a certain period of time, serve the purpose; whether they be of the same sign and amplitude or not, they can establish if the two banks belong to a single geological block or to two different but continuous geological formations.

While on this subject of clinographic observations it may be recalled that such observations have shown the importance of the displacement of a dam due to the diurnal variation of temperature, which until now was not taken into account as were the seasonal variations which are of larger amount. The fact that the daily temperature observations are not of any importance in the case of a layer of concrete larger than 20-30 cm., does not signify that the structure does not suffer displacements due to differential heating; the displacements are maximum when the basin is full or in course of filling up; or on clear limpid days when sunshine is continuous, or when the temperatures are high. Thus the diurnal displacements are due, not so much to the action of the temperature, but that of solar radiation.

The amplitude of oscillation of the daily waves is, from what we have stated above, associated, on one side, with the physical characteristics of the dam—rock system (elasticity, temperature, etc.), with the state of influx or efflux, and finally with the meteorological conditions (duration and intensity of the solar radiation, which in its turn depends on the season of the year, the clearness of the atmosphere, etc.). When all these conditions, at least the principal ones, are normal (a point that may be easily checked from our routine measurements with the addition of a phaniometer for the measurement of solar radiation), it follows that the amplitude of oscillation of the diurnal waves is due to the ratio between the moduli of elasticity of rock and concrete. This ratio, as is well known, has a great importance at the stage of designing the dam and it is advisable to check its value periodically. Further investigations on this subject are now in progress, and we expect that the results will be of some interest.

The great sensitiveness of the clinographs permits us to observe, not only the deformations, elastic or not, due to seismic effects more or less associated with the succession of the influxes and effluxes or the displacements due to the direct action of the latter as well as those due to thermal action. Clinographic observations permit us also to observe the permanent deformation due to the mutual adjustment of the dam-rock system and also the plasto-viscose deformations due to the slow and small scale, and in a certain sense continuous changes in the microstructure of the dam and the rock. More specially, when we have to deal with a fissured rock, it may happen, depending on the hydrostatic load, that a contraction is effected and consequently a partial plastic closing of the numerous fissures of the rock block. When the pressure ceases to act, these small fissures are formed again, or are only formed in a reduced degree. The opposite phenomenon may also happen, namely an increase in the porosity of the rock (understood in a broad sense) due to the innumerable small lesions or minute fractures originated by the agitations to which the rock is subjected over a long period in consequences of the small microseisms which as stated, arise during inflow*.

As regards the concrete, it is well known that its modulus of elasticity increases with time, but that this increase, for structures under a load on even partially or temporarily loaded (as dams are) is marked by the so-called phenomenon of plasto-viscous flow.

Investigations about the modulus may be made with specimens of the concrete, but more reliable results are obtained by means of small artificially induced seisms†, or with ultrasonics. This method permits, by measuring the velocity of propagation of the elastic waves and of the density of the medium to obtain a very exact idea of the variations of the elastic modulus. The same method is also employed successfully for investigating the variations of the elastic modulus of the rock beds.

Further the alternation of the maxima and minima in the amplitude of the seisms determines variations of pressure on the bottom, which are transmitted through the structure in the form of small variations of the inclinations, and these variations are recorded by the clinographs. The amplitude and the period of the oscillations recorded differ from the actual values and the record made of the movements of water does not tally exactly, that is, it does not synchronise with the movements, especially as regards the period. This is because the hysterisis in the variation of the vertical component in the two directions is not equal (the structure generally reacts immediately to the effects of a new force, but with a certain retardation to the cessation of the new force).

It has been observed by Caloi‡ that there is an important correlation between an abnormal preliminary activity of the clinograph records and subsequent abnormal macroseismic or micro-seismic activity. In fact, gradually increasing continuous movements in the different strata of the rock bed, caused by the tensions to which it is subjected, would lead to a veritable state of unstable equilibrium (earthquake) and result (in the final step) into a sudden rupture

* Pietro Caloi—Maria Cecilia Spadea—"Decrease of the value of the elastic/modulus in rocks in contact with artificial water reservoir." Ann. Geofis VII (1954). No. 4.

† Pietro Caloi—"Periods of free oscillation of the voussoir of a dam, and their relations with the elastic characteristics of concrete." Energ Elether XXXIII (1956) No. 12.

‡ Pietro Caloi—Maria Cecilia Spadea "Relations between slow variations of the inclination and seismic movements in highly seismic regions" Accad. Naz. Lincei—R. C. Sc. Fis. Mat—XVIII (1955) p. 251.

of the rock. As an example, the variations in rotation observed from September 1954 in the foundation rock of Ambiesta (Tolmezzo) Dam which is now under construction are of great interest. The rotation is at first in the N direction, but later it deflects towards W in which direction it was especially active during the period 3 to 8 October. During this period the first seisms detectable by the instruments commenced and continued with increasing frequency. This confirmed that there was tension in the ground which was in slow movement.

Subsequently the rotation again changed its direction to N, and corresponding to this change of direction there was on 11th October a shock of the order of $10^{18}$ erg, followed by an almost uninterrupted series of small instrumental shocks. The latter had the function of exhausting the residual tensions which remained in the strata involved in the earthquake.

Another new point on this subject is the verification that the maximum velocity of seismic wave is generally associated with a maximum in the frequency. This is an important generalisation of a phenomenon already known in other fields of physics. This phenomenon affords a new quantitative basis for the classification of the elastic characteristics of a solid medium: the latter is the more compact the greater the frequency of the seismic waves that it can conduct provided of course all other conditions are the same*. Our knowledge on this point of seismology may be further increased by the geophysical interpretation of the displacements of the bench marks of a high precision trigonometric survey (such as employed for the checking of the behaviour of a dam). These displacements permit us to obtain the ellipsoid of the elastic deformations of the point under consideration.

By way of conclusion we wish to mention another interesting result, still unpublished, obtained very recently by Prof. Caloi. It is known that large arch dams in Italy are not directly "encased" in the foundation rock but are supported as it were on a cushion solidary with the rock. This cushion has, in the lower and central part of the river bed, a special feature (tampon) so disposed as to render continuous the structure, which thus acquires the possibility of reacting, at least within certain limits, to the actions for which it has been designed (hydrostatic load and thermal forces) independently of the actions which proceed from the impost and the foundations. The latter actions are not easily determinable, but which are absorbed by the cushion and the tampon.

This scheme and the tests made with models on the efficacy of the perimetral joint which is established between the cushion and the structure were confirmed very satisfactorily be measurements made on the Ambiesta Dam. Measurements were taken by means of two clinographs installed in the principal cross-section of the dam: (1) at the upper limit of the tampon, and the other (2) one metre higher at the lower limit of the structure, that is above and below the perimetral joint. These measurements show that the structure reacts, for the most part, to the actions of the hydrostatic load, while

* Pietro Caloi—"Dispersion of seismic waves in the range of very high frequencies." Ann. Geofis (1957) No. 3, 4.

the tampon is sensitive to the thermal action, especially that of the diurnal variation of temperature, which is chiefly manifested in the form of long diurnal waves both in the direction upstream and downstream and in the direction abutment-abutment. The diurnal up-and-downstream wave is, as already stated, caused by the radiation of the tampon, while the abutment-abutment wave is caused by the difference of the radiation effects on the cushion and the rock. It is seen that the tampon has an elliptical movement during the day, while the movement of the structure is nearly linear. These are small displacements of the order of the tenth of a second.

This brief memorandum shows how much seismology has contributed directly and indirectly to investigations on the design and security of dams. The applications of seismology, we have described, are due to the suggestions of Prof. Caloi and his collaborators. They lead to the conclusion, which though not new, is still of special importance, that structure and ground formed a single complex, in which each of the two elements intervenes with actions and reactions which interact with each other in a complex way, but not always immediately. Therefore only continuous and contemporaneous investigations on both the elements can contribute to clarify the various interesting problems which present themselves whether to dam builders or to students of geophysics.

# BIBLIOGRAPHY ON EARTHQUAKE ENGINEERING

## Earthquake and General Characteristics

1. Earthquake Investigations in California, U.S. Department of Commerce, Coast and Geodetic Survey 1934-35.
2. Davison, C. Studies on the Periodicity of Earthquakes. page 99.
3. Mem. Geol. Sur. Ind. XXIV (1809). Assam Earthquake of 1887.
4. Mem. Geol. Suv. Ind., 73, (1939). Bihar & Nepal Earthquake with a discussion in chapters 8 & 9 on the geological structure in Northern India in relation to earthquake.
5. W. D., West. Earthquakes in India, Presidential Address. Indian Science Congress 1937, with a similar discussion.
6. Middlemiss C. S. (1910). The Kangra Earthquake on 4th April, 1905. Mem. G.S.I. Vol. 38.
7. (1939) The Bihar Nepal Earthquake of 1934 by Officers of the Geological Survey of India. Mem. No. G.S.I. Vol. 73.
8. Berkey C. P. & Nickell F. A. (1939). Report on Seismic Activity at Boulder Dam—U.S. Bureau of Reclamation, Department of Interior Denver Colorado (unpublished).
9. Gutenberg, B. & Richter C. F. Earthquake Magnitude, Intensity, Energy and Acceleration. Bulletin of the Seismological Society of America, Vol. 46 No. 2, April 1956.
10. Hershberger. J. Comparison of Earthquake Acceleration with Intensity Ratings. Bulletin of the Seismological Society of America, Vol. 46, No. 4, Oct. 1956.
11. Anderson A. W. and others. Lateral Forces of Earthquake & Wind. Paper No. 2514, Transactions, Am. Soc. C.E., Vol. 117, 1952, page 716.
12. Gutenberg B. Effects of Ground on Earthquake Motion. Bulletin of the Seismological Society of America, Vol. 47, 1957.
13. Housner, G. W. R. R. Martel & S. L. Alford. Spectrum Analysis of Strong Motion Earthquake. Bulletin of the Seismological Society of America, Vol. 43, 1953.
14. Byerly, P. Seismology, Prentice-Hall, New York, 1942.
15. Carder, D. S. and Gilmore, M. H. Ground Vibrations. Seismological Society of America, Bulletin, Vol. 35, No. 1, Jan. 1945.
16. Cornwell, F. E. Correlation Between Various Mothods of Indicating Earthquake Intensities, (informal memorandum). U.S. Bureau of Reclamation, Aug. 2, 1946.

17. Gutenberg, B. Amplitudes of Surface Waves and Magnitudes of Shallow Earthquakes, Seismological Society of America, Bulletin Vol. 35, No. 1, Jan. 1935.

18. Gutenberg, B., Amplitudes of P, PP and S and Magnitude Seismological Society.

19. Gutenberg, B., Amplitudes of P, PP and S and Magnitude of Shallow Earthquakes, Seismological Society of America, Bulletin, Vol. 35-36, 1945-46, No. 2, April 1945. p. 57.

20. Gutenberg, B., and Richter, C. F. Earthquake Magnitude, Intensity, Energy, and Acceleration, Seismological Society of America Bulletin, V. 32, No. 163, p. 191, 1942.

21. Heck, N. H. Earthquakes, Princeton University Press, 1936.

22. Richter, C. F. An Instrumental Earthquake Magnitude Scale, Bulletin Seismological Society of America, V. 25, pp. 1—25, 1935.

23. Ulrich, F. P. Reporting the Northwest Earthquake from the Scientific Point of View, Building Standards Monthly, p. 8, June 1949.

24. Earthquake Hazard and Earthquake Protection. Joint Technical Committee on Earthquake Protection, Los Angeles, California, June 1933.

25. Physics of the Earth. Bulletin, National Research Council, Vol. VI, Seismology, No. 190, Oct.

26. Gulatee, B. L. Geodetic and Geophysical Aspects of the Earthquakes in Assam. Central Board of Geophysics (India), Publ. No. 1, pp. 16—25 (1953).

27. Bullen, K. E. Introduction to the Theory of Seismology. 296 pp., 2nd ed., Cambridge Univ. Press (1953).

28. Milne, John; Lee, A. W. Earthquakes and other Earth Movements. 244 pp., 7th ed., Blakiston (1939).

29. Earthquake Investigation in the United States, prepared by Frank Neumann, Special Publ. No. 282, revised 1953, 40 pp.

30. Housner, G. W. A Dislocation Theory of Earthquakes. 34 pp., Eq. Res. Lab., Calif. Inst. Tech., 4th Tech. Rep. to ONR (1953).

31. Properties of Strong Ground Motion Earthquakes. Bull. SSA, 45 : 197—218 (1955).

32. Neumann, Frank. See also Wood and Neumann. Earthquake Intensity and Related Ground Motion.

33. Central Board of Geophysics, India A Compilation of papers on the Assam Earthquake of Aug. 15, 1950.

34. Hudson, D. E. A Preliminary Report on A Simplified Strong-Motion Earthquake Recorder. 6 pp., Eq. Res. Lab., Calif. Inst. Tech., Jan. 1958.

35. Nazarov, A. G. Instrumental Determination of Seismic Forces for Design of Structures (in Russian) Bull. Acad. of Sci., Armenian S.S.R., No. 3, pp. 31—50 (1947).

36. Davison, Charles. The Japanese Earthquake of 1923. 127 pp., Thomas Murby & Co., London 1931.

37. Eaton, J. Edmund. Long Beach, Calif., Earthquake of March 10, 1933. Bull. AAPG, 17 : 732—738, 1933.

38. Garfias, Valentin R. Chapin, Theodore C. Geology of Mexico. 202 pp. 1949.

39. Maso, Miguel Sanderra; Smith Warren D. The Relation of Seismic Disturbances in the Philippines to the Geologic Structure. Bull. SSA, 3 : 151—186 (1913).

40. Trask, Parker, D.; Rolston, Jack W. Engineering Geology of San Francisco Bay, California. Bull. GSA, 62 : 1079—1109, 1951.

41. Wadia, D. N. Geology of India. 531 pp., 3rd ed. Macmillan, 1953.

42. Weaver, Charles E. Geology of the Coast Ranges Immediately North of the San Francisco Bay Region, Calif. GSA Memoir 35, 242 pp. (1949).

43. Allen, Clarence R. San Andreas Fault Zone in San Gorgonio Pass, Southern Calif. Bull. GSA, 68 : 315—349, 1957.

44. Arnold, Ralph. Topography and Fault System of the Region of the San Jacinto Earthquake. Bull. SSA, 8 : 68—73, 1918.

45. Crowell John C. Strike-slip Displacement of the San Gabriel Fault. Southern Calif. Chap. IV, paper No. 6. pp. 49—52, Calif. Div. Mines Bull. 170 (1954).

46. Gianella Vincent P. Earthquake and Faulting. Fort Sage Mountains Calif. Dec., 1950. Bull. SSA, 47 : 173—177 (1957).

47. Gianella Vincent P. Earthquakes and Surface Faulting in the Great Basin. Paper presented at meeting, Seismological Soc. of America, Univ. of Calif, Los Angeles, April 1957, (not yet published).

48. Hill, Mason L.; Dibblee, T. W. Jr. San Andreas, Garlock, and Big Pine Faults. Calif, Bull. GSA, 64 : 433—458, 1953.

49. Hill, Mason L. Tectonics of Faulting in Southern Calif. Chap. IV, paper No. 1, pp. 5—13, Calif. Div. Mines Bull. 170, 1954.

50. Louderback, G. D. Characteristics of Active Faults in the Central Coast Ranges of Calif., with application to the safety of dams. Bull. SSA, 27 : 1—27 (1937).

51. Noble, Levi F. The San Andreas Fault Zone from Soledad Pass to Cajon Pass, Calif. Chap. IV, paper No. 5, pp. 37—38, Calif. Div. Mines Bull. 170 (1954).

52. Noble, Levi F. The San Andreas Rift and Some Other Active Faults in the Desert Region of South-Eastern Calif. Report of Adv. Comm. on Seismology. Reprinted from Carnegie Institution of Washington Year Book No. 25, (1925-1926) in Bull. SSA, 17 : 25—39, 1927.

53. Taber, Stephen. Some Criteria Used in Recognising Active Faults. Bull. GSA, 34 : 661—668 (1923).

54. Davison, Charles. Great Earthquakes. 286 pp. Thomas Murby & Co., London (1936).

55. Far East Command, Gen. Hdqtrs. The Fukui Earthquake, Hokuriku Region, Japan, 28 June 1948. Vol. 1, Geology, 81 pp.

56. Grant, U.S. Subsidence and Elevation in the Los Angeles Region. pp. 129—158 of Science in the Univ., publ. by the Univ. of Calif. Press (1944).

57. Grant, U.S.; Sheppard, W.E. Some Recent Changes of Elevation in the Los Angeles Basin of Southern California, and their Possible Significance. Bull. SSA, 29 : 299—326, 1939.

58. Hayford, J. F.; Baldwin, A. L. Geodetic Measurements of Earth movements. See Calif State Earthquake Investigation Commission.

59. Meade, B. K. Earthquake Investigations in the Vicinity of El Centro, Calif.; Horizontal Movement. Trans. AGU, 29 : 27—31 (1948).

60. Whitten, C. A. Crustal Movement in Calif. and Nevada. Trans. AGU, 37 : 393—398, 1956.

61. Whitten, C. A. Horizontal Earth Movement, Vicinity of San Francisco Calif. Trans. AGU, 29 : 318—323, 1948.

62. Byerly, Perry. Pacific Coast Earthquakes. 38 pp. Condon Lectures in Oregon (1952).

63. Byerly, Perry. Seismology. 256 pp. Prentice-Hall (1942).

64. Davison, Charles. Earthquake Sounds. Bull. SSA, 28 : 147—161, 1938.

65. Davison, Charles. The Founders of Seismology. 240 pp., Cambridge Univ. Press (1927).

66. Davison, Charles. Volcanic Earthquakes. Jour. Geol., 29 : 97—124, 1921.

67. Eiby, G. A. About Earthquakes. 168 pp., Harper (1957).

68. Heck, N. H. Earthquakes. 222 pp., Princeton Univ. Press (1936).

69. Hollis, Edward P. The Earthquake Problem in the Western Hemisphere Bull. Pan-American Union, Washington, D.C., Feb. & March. 1948, pp. 86—100 & 147—150.

70. Kawasumi, Hirosi. Seismology in Japan, 1939—1947. Bull. SSA, 39 : 157—167, 1949.

71. Leet, L. D. Practical Seismology and Seismic Prospecting. 430 pp. Appleton-Century (1938).

72. Macelwane, James B. When the Earthquakes. 288 pp., The Bruce Publishing Co., Milwaukee, Wisconsin, 1947.

73. Macelwane, James B. The Structure of the Interior of the Earth. Trans. Kansas Acad. of Sci., Vol. 51, No. 2, pp. 143—161, June, 1948.

74. McAdie, Alexander G. Japan's Contribution to Seismology. Bull. SSA, 2 : 153—160 (1912).

75. Montessus de Ballore, F. de. The Seismological Work of John Milne. Bull. SSA, 4 : 1—24, 1914.

76. National Research Council Bull. 90. Physics of the Earth, Vol. VI : Seismology, 223 pp. (1933).

77. Sieberg, A. Earthquake Research. 144 pp., Gustav Fischer, Jena, 1933.

78. Terada, Terahiko; Matuzawa, Takeo. A Horizontal Sketch of the Development of Seismology in Japan. Chap. XIII, pp. 351—410, of Scientific Japan.

79. Wood, Harry O. The Earthquake Problem in the Western United States. Bull. SSA, 6 : 197—217 (1916).

80. Allen, C.R., St. Amand, P.; Richter C.F. Geologic Structure and Seismicity in Southern Calif. and Adjacent Areas. March 1958.

81. Broggi, J. A. Preliminary Note on a Physical Phenomenon Resembling Mountain Building. Jour. Geol., 43 : 1067—1070 (1935).

82. Bucher Walter H. The Crust of the Earth. Scientific American, Vol. 182, No. 5, pp. 32—41, May 1950.

83. Bucher Walter H. The Deformation of the Earth's Crust, An Inductive Approach to the Problems of Diastrophism. 518 pp.; Princeton Univ. Press (1933).

84. Bucher Walter H. Deformation in the Organic Belts. GSA special paper 62, pp. 343—368 (1955).

85. Bullard, Sir Edward. The Interior of the Earth. See Kuiper.

86. Daly, R. A. Strength and Structure of the Earth. 434 pp., Prentice-Hall (1940).

87. Gilluly, James. Distribution of Mountain Building in Geologic Time. Bull. GSA, 60 : 561—590 (1949).

88. Griggs, David. A Theory of Mountain-Building. Am. Jour. Sci., 237 : 611—650 (1939).

89. Gutenberg, B. Internal Constitution of the Earth. Physics of the Earth, Vol. VII, National Research Council; B. Gutenberg, Editor; 1st. ed., McGraw-Hill 1939; 2nd ed. Dover Publ. 439 pp., 1951.

90. Gutenberg, B.; Richter, C. F. Seismicity of the Earth. 310 pp., 34 maps, 2nd ed., Princeton Univ. Press, 1954.

91. Hapgood, Charles H.; Campbell, James H. Earth's Shifting Crust. 438 pp., Pantheon (1958).

92. Hubbert M. King. Theory of Scale Models as Applied to the Study of Geologic Structures. Bull. GSA, 48 : 1459—1520 (1937).

93. Koch, Thomas W. Analysis and Effects of Current Movements on an Active Fault in Buena Vista Oil Field, Kern Country, Calif. Bull. AAPG, 17 : 694—712 (1933).

94. Leuderback, G. D. Faults and Earthquakes. Bull. SSA, 32 : 305—330 (1942).

95. Tocher, Don. Earthquake Energy and Ground Breakage. Bull. SSA, 147—153, 1958.

96. Umbgrove, J. H. F. The Pulse of the Earth. 358 pp.. 2nd ed., Martinus Nijhoff, The Hague, 1947.

97. Umbgrove, J. H. F. Structural History of the East Indies. 63 pp., Cambridge Univ. Press, Am. Branch, New York, 1949.

98. Vening Meinesz, F. A. Plastic Buckling of the Earth's Crust. GSA, Special paper 62, pp. 319—330 (1955).

99. Benioff, Hugo. Earthquakes and Rock Creep (Part 1 : Creep characteristics of rocks and the origin of aftershocks). Bull. SSA, 41 : 31—62 (1951).

100. Byerly, Perry. Earthquake Origin and Propagation. Proc. Symposium on earthquakes and blast effects on structures, pp. 1—7.

101. Byerly, Perry. Nature of Faulting as Deduced from Seismograms GSA, special paper No. 62, pp. 75—85, 1955.

102. Conrad, V. Trigger Causes of Earthquakes and Geological Structures. Bull. SSA, 36 : 357—362, 1946.

103. Davison, Charles. The 42-Minute Period in the Frequency of Aftershocks. Bull. SSA, 23 : 57—79, 1933.

104. Davison, Charles. Studies on the Periodicity of Earthquakes. 107 pp., Murby, London, 1938.

105. Macelwane, J. B. Forecasting Earthquakes. Bull. SSA 36 : 1—4, 1946.

106. Reid, Harry Feilding. The Elastic-rebound Theory of Earthquakes. Univ. of Calif. Bull. Deptt. of Geol. 6 : 413—444, 1911.

107. Sassa, Kenzo; Nichimura, Eiichi. On Phenomena Forerunning Earthquakes Disaster Prevention. Res. Intt., Kyoto, Univ. Kyoto; Bull. No. 13, pp. 1—8, Feb. 1956.

108. Heck, N. H. A New Map of Earthquake Distribution, Geographical review, 25; 125—130, 1935.

109. Oldham, Richard D. Report on Great Earthquake of 12 June, 1897. Geological Survey of India, Memoirs, Vol. XXIX, 379 pp., 1899.

110. Den Hartog, J. F. Mechanical Vibrations. 436 pp., 4th ed., Mc-Graw-Hill (1956).

111. Freberg, C. R.; Kemler, Emory N. Elements of Mechanical Vibrations. 417 pp., Wiley (1952).

112. Timoshenko, Stephen P. Vibration Problems in Engineering 468 pp., 3rd ed., Van Nostrand (1955).

113. Bruce, V. G. A Graphical Method for Solving Vibration Problems of a Single Degree of Freedom. Bull. SSA, 41; 101—108 (1951).

114. Den Hartog, J. P. The Amplitudes of Non-Harmonic Vibrations. Jour. Franklin Institute, 216 : 459—473 (1933).

115. Benioff, Hugo. Earthquakes Seismographs and Associated Instruments. pp. 219—275, Vol. II of Advances in Geophysics, Academic Press (1955).

116. Benioff, Hugo. The Physical Evaluation of Seismic Destructiveness : Bull. SSA, 24 : 398—403 (1934).

117. Blanchard, F. B.; Byerly Perry. A Study of a Well Gauge as a Seismograph. Bull. SSA, 25 : 313—322 (1935).

118. Carder, Dean S. Response of a Torsion Seismometer to Sustained Vibrations. Bull. SSA, 30 : 343—352 (1940).

119. Davison, Charles. On Scales of Seismic Intensity and on the Construction and Use of Isoseismal Lines. Scales of Seismic Intensity : Supplementary Paper. Bull. SSA, 23 : 158—166 (1933).

120. Heck, N. H. A Review of Outstanding Problems in Strong Motion Vibration work. Bull. SSA, 25 : 343—348 (1935).

**Engineering Aspects**

1. Westergaard H. M., Water Pressures on Dams During Earthquakes, Transactions, American Society of Civil Engineers. 98 : 418—472 (1933).
2. Dewell, H. D., Earthquake Resistant Construction, Engineering News Record, April 1928, page 650.
3. Fergneson, F. F. Earthquake Effects on Dams. Central Board of Irrigation Journal, May 1950.
4. Guha, S. K. & Ram Gurdas, Vibration of Dams—Their Free Period, Central Board of Irrigation Journal, July 1950.
5. Housner G. W. Limit Design of Structures to Resist Earthquakes Proceedings of the World Conference on Earthquake Engineering, 1956.
6. Jai Krishna. The Earthquake Engineering Problems in India, California, Institute of Technology, Pasadena, California, April 1957.
7. Berg G. V. Analysis of Structural Response to Earthquake Forces. Report I.P.-291, University of Michigan, Engineering College Industry Programme, May 1958.
8. Bergstrom S. G. and Linderholm, S. Dynamic Method for Determining Average Elastic Properties of Surface Soil Technical University, Stockholm.

9. Bernhard, R. K. Geophysical Study of Soil Dynamics, Technical Publication 834, American Institute of Mining and Metallurgical Engineers, 1938.

10. Bruggeman, J. R., Zangar, C. N. and Brahtz, J. H. A. Notes on Soil Mechanics, Technical Memorandum No. 592, U.S. Bureau of Reclamation, June, 1939.

11. S. L. Kumar, Director, Railway Research Centre, Lucknow. Earthquake Resisting Construction in India, Earthquake Engineering Seminar, University of Roorkee, 1959.

12. Casagrande, A. and Shannon, W. L. Strength of Soils Under Dynamic Loads, American Society of Civil Engineers, Proceedings, Vol. 74, p. 591, April 1948.

13. Daehn W. W. The Slip Circle Method of Stability Analysis, Treatise on Dams ch. 8, Earth Dams, Appendix A, p. 85, U.S. Bureau of Reclamation, May 1948.

14. Furkert, F. W. Effect of Earthquake on Engineering Structures. Institute of Civil Engineers, No. 236, pp. 344—68, 1934.

15. Hilf, J. W. Estimating Construction Pore Pressures in Rolled Earth Dams, Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. III, Rotterdam, June 1948.

16. Hruban, K. The Influence of the Heterogeneity of the Soil Mass on its Deformation. Applied Mechanics, rew. Jan. 1949.

17. Jacoboen, L. S. Motion of a Soil Subjected to a Simple Harmonic Ground Vibration. Seismological Society of America Bulletin, No. 20, 1930.

18. Lyman, A. K. B. Compaction of Soils by Explosives. Civil Engineering, V. 10, No. 4, pp. 205—207, New York, April 1940.

19. Matel R. R. Effect of Earthquake on Earth Dams. Military Engineer, V. 30, No. 173, p. 359, Sept.-Oct., 1938.

20. Melville, F. R. and Hetherington. R. LeG. The Pegu Yomas Scheme for the Water Supply of Rangoon. Paper 5593, Journal of the Institute of Civil Engineers, No. 5, 1947-48, March 1948.

21. Mononobe, N., Takata, A., and Matumura, M. Seismic Stability of the Earth Dam. Question VII, Second Congress on Large Dams, Washington, D.C., 1936.

22. Morris, S. P., and Pearce, C. E. Earthquake Forces on Dams. Bulletin Seismological Society of America, Sept. 1931.

23. Newell, C. S., and Sollid, E. Method of Calculating Deflecttion of a Structure Subjected to a Known Earthquake. U.S. Bureau of Reclamation, Technical Memo. 583, Nov. 1938.

24. Parsons, A. W. Accelerated Settlement of Embankments by Blasting Public Roads. V. 20, No. 10, pp. 197—202, Dec. 1939.

25. Richardson, J. T. A Summary of Seismological Investigations Made in the Lake Mead Area, with Comments on Data Obtained at the Shasta and Grand Coulee Dams. U.S. Bureau of Reclamation, Technical Memo. 634, April 1947.

26. Rose, E. D. Inelastic Structural Action in Earthquake Resistant Design. Informal Memo. U.S. Bureau of Reclamation, Dec. 10, 1943.

27. Tibbetta, F. H. Earthquake Prood Earth Dams. Engineering News-Record, July 2, 1936.

28. Tschebotarioff, G. P., and Ward. E. R. Resonance of Machine Foundations and the Soil Coefficients which Affect it. Foundation Engineering, Proceedings, Mechanics, V. 1, pp. 309—313, 1948.

29. Waldorf, W. A., and Newell, C. S. Energy Method of Calculating Natural Vibration Periods of Structures as Applied to Dams. U. S. Bureau of Reclamation, Technical Memo. 585, Sept. 1939.

30. Wilson, G., and Sutton, J.L.E. A Contribution to the Study of the Elastic Properties of Sand. Foundation Engineering, Proceedings, Second International Conference of Soil Mechanics, V. 1, pp. 197—202, 1948.

31. Zangar, C. N. Hydrodynamic Pressures upon Dams Due to Horizontal Earthquake Shock. Informal Memorandum, U.S. Bureau of Reclamation, April 20, 1950.

32. Dams Tested by Artificial Earthquakes. Engineering News Record, p. 184, Aug. 11, 1938.

33. Forced Vibration Tests. Morris Dam, U.S. Coastuand Geodetic Survey, California Seismological Programme Preliminary Report.

34. Report on Seismological Studies of Baker River Dam. U.S. Coast and Geodetic Survey, Seismological Field Survey, Preliminary Report SP-12.

35. Three Dams on San Andreas Fault have Resisted Earthquakes. Engineering News Record, Aug. 25, 1932.

36. Nagaho Mononobe: Characteristics and Rationalised Design Methods of Gravity Dams for Water Storage. Journal of the Japan Society of Civil Engineers. Vol. II, No. 5, 1922.

37. Nagaho Mononobe: Stability of Reinforced Concrete Buttress Dams During Earthquakes. Journal of Seismological Society of Japan, No. 5, 1928.

38. Nagaho Mononobe: Method of Determining the Standard Section of Gravity Dams Taking into Account Dynamic Pressure Caused by Earthquakes. Public Works Research Institute Report No. 26, 1934.

39. Magoji Matsumura: Deformation of Earth Dams During Earthquakes. Public Works Research Institute Report No. 28, 1934.

40. Measurement of Rigidity Coefficients of Earth Dams. Akira Takada Civil Engineering No. 5-3-1936 of the Seismological Society of Japan, No. 14-12-1942.

41. Fuyuhiko Gishinoue. Vibration Measurements of Gravity Dams During Earthquakes. Journal of the Seismological Society of Japan, No. 14-12-1942.

42. Seiichi Sato. Dynamic Water Pressure Acting on Dams During Earthquakes. Kensetsu-In Dainigijutsu Kenkyujo Gaiho No. 3, 1948.

43. Tadashi Hatano. Seismic Force Effect on Gravity Dams (No. 1, 2 and 3). Transaction of the Japan Society of Civil Engineers No. 3—5, 1949-50.

44. Tadashi Hatano. Seismic Force Effect on Gravity Dams (No. 3). Journal of the Japan Society of Civil Engineers Vol. 35, No. January 1950.

45. Tadashi Hatano. Vibration Tests of Gravity Dams with Agaragar Models. Journal of the Japan Society of Civil Engineers Vol. 36, No. 10, Oct. 1951.

46. Tadashi Hatano. Model Study of Dynamics Water Pressures on Gravity Dams During Earthquakes. Journal of Japan Society of Civil Engineers, Vol. 36, No. 11, Nov. 1951.

47. Tadashi Hatano. Influences of Earthquakes on Gravity Dams & Kiyoshi Tanaka: Journal of the Japan Society of Civil Engineers, Vol. 37, No. 3, 1952.

48. H. Masuda and T. Takahashi. Study of Seismic Factors for Design of Dams in Japan. Monthly Journal of the Electric Power Research Laboratory, Vol. 3, No. 3, 1952.

49. Tadashi Takasashi. On the Characteristics of Earthquake Motion at Damsite and the Response of Structures. Monthly Journal of the Electric Power Research Laboratory, Vol. 4, No. 5, 6, 1954.

50. T. Takahashi. Experimental Study on the Period of Natural Vibration of Earth Dams. Monthly Journal of the Electric Power Research Laboratory, Vol. 2, No. 21, 31, 1952.

51. T. Takahashi. A Method of Determination of the Period of Natural Vibration of Earth Dams, and a Supplement. Monthly Journal of the Electric Power Research Laboratory, Vol. 2, No. 21, 31, 1952.

52. T. Takahashi. A Method of Determination of the Period of Natural Vibration of Arch Dams. Journal of the Electric Power Research Laboratory, Vol. 3, No. 1, 1953.

53. Experimental Researches of Arch Dam—Arch Dam Committee of the Tokyo Electric Power Co. Inc. 1955.

54. Motohiro Hatanka. Stability of Arch Dam During Earthquakes. Disaster Prevention Research Institute, Kyoto Univy.

55. Motohiro Hatanka. Three Dimensional Consideration on the Vibration of Earth Dams—on the Free Vibration. Journal of the Japan Society of Civil Engineers, Vol. 37, No. 10, Oct. 1952.

56. Hiroshi Minami. Stability Section of Earth Dam During Earthquakes. Symposium of Earthquake Engineering in Japan Society of Civil Engineers, 1958.

57. Y. Yokoo, H. Ishizaki and N. Hatakeyama. Vibration Characterstics of Earth Dams. Transactions of the Japan Society of Civil Engineers, No. 49, Oct. 1957.

58. Report on the Vibration Studies of Ojiya Dam : National Railways Technical Research Institute Report No. 7—114, 1956.

59. Y. Niwa and C. Mori. Studies on the Earthquake Resistant Properties of Earth and Rock-fill Dams. Transactions of the Japan Society of Civil Engineers, No. 58, Sept. 1958.

60. Yoshiji Niwa. On the Distribution of Stress in a Hollow Gravity Dam During an Earthquake in the Direction of the Dam Axis. Preprint for the 11th Annual Meeting Japan Society of Civil Engineers, 1956.

61. Vibration Model Test of Ikawa Hollow Gravity Dam. Journal of Technical Laboratory, Vol. 7, No. 6, 1957.

62. T. Hatano, T. Takahashi and H. Tsutsumi. Results of Vibration Test on Tsukabaru Gravity Dam and its Consideration. Transactions of the Japan Society of Civil Engineers. No. 59, 1958.

63. Motohiro Hatanaka. Earthquake Resistant Design Features for Concrete Dams. Preprint for the 14th Annual Meeting Japan Society of Civil Engineers, 1959.

64. Seima Kotsubo. Dynamic Water Pressures on Arch Dams During Earthquakes. Transactions of the Japan Society of Civil Engineers, No. 44, April 1957.

65. Seima Kotsubo. Model Experiment of Dynamic Water Pressures on Arch Dams. Transactions of the Japan Society of Civil Engineers, No. 46, June 1957.

66. Seima Kotsubo. Dynamic Water Pressure on Dams due to Irregular Earthquakes. Transactions of the Japan Society of Civil Engineers, No. 47, Aug. 1957.

67. Seima Kotsubo. Stability of Gravity Dams During Earthquakes. Transactions of the Japan Society of Civil Engineers, No. 55, May 1958.

68. Tadashi Takahashi. Vibration Characteristics of Kamishiiba Dam. The Symposium on the Earthquake Engineering of Japan Society of Civil Engineers, No. 3, 1959.

69. S. Okamoto, K. Kato, M. Hakuno and T. Arakawa. On the Elastic Vibration of Arch Dams. The Symposium on the Earthquake Engineering of Japan Society of Civil Engineers, No. 3, 1959.

70. Dynamical Compressive Deformation and Failure of Concrete under Earthquake Load. Transactions of the Japan Society of Civil Engineers, No. 57, 1960.

71. Arch Dam Committee of the Tokqo Electric Power Co. Inc., Experimental Researches of Arch Dam, C-12, 5th Congress of ICOLD, 1955.

72. T. Hatano: "The Stability of an Arch Dam against Earthquake Vibration Test and their Studies on Naramata Arch Dam", Technical Report C. 5607, Technical Research Laboratory, Central Research Institute of Electric Power Industry, 1957.

73. M. Hatanaka: "Fundamental Considerations on the Earthquake Resistant Properties of the Earth Dam", Disaster Prevention Research Institute, Kyoto University, Bulletin No. 11, 1955.

74. Y. Niwa and C. Mori: "Experimental Study on the Earthquake Resistant Properties of Earth and Rock-fill Dams". Technical Reports of the Engineering Research Institute, Kyoto University, Vol. VIII, No. 6, 1958.

75. M. Hatanaka, Y. Niwa and S. Samukawa: "Researches on Vibration Characteristics of Hollow Gravity Dams", C-4, 6th Congress of ICOLD, 1958.

76. T. Hatano: "Dynamic Study of Tsukabaru Gravity Dam", Technical Report C. 5801, Technical Research Laboratory, Central Research Institute of Electric Power Industry, 1958.

77. M. Hatanaka: "Fundamental Studies on the Aseismic Design of Gravity Dams, Memoirs of the Faculty of Engineering" Kobe University, No. 5, March 1958.

78. S. Okamoto: "Small Scale Model Test on Arch Dam", Proc. of 8th Japan National Congress for Applied Mechanics, 1958.

79. Bilt, M. A. Analytical and Experimental Methods in Engineering Seismology. Trans. ASCE, 108: 365—408 (1943).

80. Creskoff, Jacob J. Dynamics of Earthquake Resistant Structures, 127 pp. McGraw-Hill (1934).

81. Earthquake Engg. Research Institute, Univ. of Cal. Los Angeles. The Symposium on Earthquake and Blast Effects on Structures, held in Univ. of Calif, Los Angeles, June 1952. 322 pp.

82. Frankland J. M. Effects of Impact on Simple Elastic Structures Proc., Soc. for Experimental Stress Analysis, Vol. VI, No. 2, pp. 7—27. (1948).

83. Hudson, D. C. Response Spectrum Techniques in Engineering Seismology. Paper No. 4, 12 pp., World Conference on Earthquake Engineering (1956).

84. Hudson, G. E. A Method of Estimating Equivalent Static Loads in Simple Elastic Structures. Proc., Soc. for Experimental Stress Analysis, Vol. VI, No. 2, 28—40, 1948.

85. Jacobsen, Lydik S.; Ayre, Robert S. Engineering Vibrations —with Applications to Structures and Machinery. 564 pp. McGraw-Hill, (1958).

86. Lamb, Sir Horace. Hydrodynamics. 738 pp., 6th ed., Dover Publications, (1945).

87. Melin, J. W.; Sutcliffe, S. Development of Procedures for rapid computation of Dynamic Structural Response. Univ. of Illinois Civil Engineering Studies, Structural Research Series No. 145, Urbana, Ill. 1957.

88. Newmark, N. M. Methods of Analysis for Structures Subjected to Dynamic Loading, 67 pp. (1950). Directorate of Intelligence, U.S. Air Force Headquarters, Washington D.C.

89. Veletsos, A. S. Some Applications of a Digital Computer in Structural Research. Civil Engineering, 28 : 344—347, 1958.

90. Wilbur, John B.; Hansen, Robert J. The Importance of Dynamic Loads in Structural Design. ASCE Centennial Transactions pp. 753—766, 1953.

91. Abe, Mikishi. Tests of Reinforced Concrete Frames under Repeated Horizontal Loads and Measurement of their Periods of Free Vibration. Proc., World Engineering Congress, Tokyo, VIII : 501—559, 1929.

92. Alford, J. L.; Housner, G. W. A. Dynamic Test of a Four Storey Reinforced Concrete Building. 19 pp. Earthquake Engineering Research Institute, Aug. 1951.

93. Birkenhauer, H. F. The Prevention of Destruction from Seismic Waves. Report of the Seismological Laboratory, John Carroll Univ., Cleveland, Ohio, 92, pp. 1951.

94. Blume, John A. Structural Dynamics in Earthquake—Resistant Design. Proc. ASCE, paper 1695, 45 pp., July 1958.

95. Brooks, Nancy Brazell; Newmark, N. M. The Response of Simple Structures to Dynamic Loads. 49 pp., Tech. Rep. to ONR Univ. of Illinois Civil Engineering Studies, Structural Research Series No. 51, Urbana, Illinois, April 1953.

96. C. Martin Duke. Masonry Structures under Blast. Report 51—15, 27 pp., Dec. 1951. Cal., Univ. of Cal., Deptt. of Engg. Los Angeles.

97. William R. Thomson and James A. Cheney. Response of Elastic Beams to Impulsive Loading. Report 51—13, 71 pp., Dec. 1951. Cal., Univ. of Cal., Los Angeles.

98. Cheney James A, Structural Analysis by Dynamic Load Parameters. Jour. ACI, 28 : 99—111, 1956.

99. DeHart, R. C. Response of a Rigid Frame to a Distributed Transient Load. Proc. ASCE, paper 1056, 23 pp., Sept. 1956.

100. Dr. Jai Krishna, Associate Professor of Civil Engineering, Roorkee. Seismic Design of Structures in India. Earthquake Engineering Seminar, University of Roorkee, 1959.

101. Evaldson, Rune L.; Ayre, Robert S.; Jacobsen, Lydik S. Response of an Elastically Non-linear System to Transient Disturbances. Jour. Franklin Institute 248 : 473—494, 1949.

102. Gershanik, Simon. Contribution al estudio de cargas sismicas sobre edificios (contribution to the study of seismic loads on structures) 49 pp., 1951, published by the Observatorio, Astronomico, Lapluti, Argentine.

103. Glover, Robert E. Theory of the Action of a Beam or Frame Subjected to an Earthquake. U.S. Bureau of Reclamation, Tech. Memo. 603, 51 pp. (1940).

104. Glover Robert E. Earthquake Stresses in Frame Structures Jour. ACI, 13 : 453—470, 1942.

105. Goodman, Lawrence E.; Rosenblueth, Emilio; Newmark, Nathan M. Aseismic Design of Firmly Founded Elastic Structures. Trans. ASCE, 120 : 782—802, 1955.

106. Goodman, Lawrence E.; Rosenblueth, Emilio; Newmark, Nathan M. Aseismic Design of elastic structures founded on firm ground. 49 pp., Tech. Rep. to ONR, Univ. of Illinois Civil Engineering Studies, Structural Research Series No. 26, Urbana, Illinois, June 1952.

107. Greaves M. J.; Mavis, F. T. Inelastic Behaviour of Impulsively loaded beams. Proc. ASCE, paper 1232, 18 pp. May 1957.

108. Hachyan, A. E. Vibration of a Cantilever Beam during Seismic Shocks taking into account the Dispersion of Energy. American S.S.R., X : No. 6, 53—66, 1957.

109. Hoskins, Leander M.; Galloway, John D. Earthquakes and Structures. Trans. ASCE, 105 : 269—322, 1940.

110. Housner, G. W. Calculating the Response of an Oscillator to Arbitrary Ground Motion. Bull. SSA, 31 : 143—150, 1941.

111. Housner, G. W. Intensity of Ground Motion during strong Earthquakes, 60 pp., Eq. Res. Lab., Calif. Inst. Tech., 2nd Tech. Rep. to ONR 1952.

112. Housner G. W.; Outinen, Hannu. The Effect of Torsional Oscillations on Earthquake Stresses Bull. SSA, 48 : 221—229, 1958.

113. Hudson, D. E.; Housner, G. W. Structural Vibrations Produced by Ground Motion. Trans. ASCE. 122 : 705—721, 1957.

114. Ibanez Valenzuela, Julio. Chilean Regulation of construction to resist earthquakes. Metting held at the Centro Argentino de Ingenierois, Sept. 15, 1942.

115. Ifrim, Mihail. Present Concepts on the Dynamical Analysis of Elastic Structures Subjected to Earthquake Motion (in Rumanian) to be published in 1958.

116. Ifrim, Mihail. Dynamic Analysis of Elastic Frames on the Basis of Conservation of Energy. (in Rumanian, with English and Russian Abstracts) VIII : No. 2, 393—406, 1957.

117. Ifrim, Mihail. An Energy Expression for the Calculation of Natural Frequency of Framed Structures. VII : No. 7, 649—655, 1957.

118. Isakson, G. Report on Survey of Analytical Methods for Determining Transient Stresses in Elastic Structures. 64 pp., 1950.

119. Jacobsen, Lydik S. An Approximate Solution of the Steady Forced Vibration of a System of one Degree of Freedom under the influence of Various Types of Damping. Bull. SSA 20 : 196—223, 1930.

120. Jacobsen, Lydik S. Experimental Study of Dynamic Behaviour of Models of Timber Walls Subjected to an Impulsive, Horizontal, Ground Vibration. Bull. SSA 20 : 115—146, 1930.

121. Jacobsen, Lydik S. Natural periods of Uniform Cantilever Beams, Trans. ASCE. 104, 402—439, 1939.

122. Jacobsen, Lydik S.; Ayre. Robert S. A Comparatvie Study of Pulse and Step-type Loads on a Simple Vibratory System. Structural dynamics tech. Rep. No. 16 to ONR, Stanford Univ., Stanford Calif., 26 pp., 1952.

123. Jacobsen Lydik S.; Jespersen, H. J. Stready Force Vibrations of Single Mass Systems with Symmetrical as well as Unsymmetrical Non-linear Resorting Elements. Jour. Franklin Instt., 220; 467—496, 1935.

124. Cole, Robert H. Underwater Explosions. 437 pp., Princeton Univ. Press, 1948, Chap. 5, pp. 147—209, deals with the measurement of underwater explosion pressures.

125. Guha, S. K.; Ram, G. Studies on Seismicity and Seismic Forces on Dams. Irrigation & Power (the Journal of the Central Board of Irrigation and Power, Simla, India), Vol. 8, No. 3, pp. 414—446, July 1951. Vol. 12, No. 1, pp. 143—161, Jan. 1955.

126. Guha, S. K.; Rao, M. G. Distribution of Horizontal Acceleration along the Height of Dam During Earthquake. Irrigation & Power (the Journal of Central Board of Irrigation and Power, Simla, India) Vol. 8, No. 4, pp. 591—599, Oct. 1951.

127. Hoskins, Leander M.; Jacobsen, Lydik S. Water Pressure in a Tank Caused by a Simulated Earthquake. Bull. SSA, 24 : 1—32, 1934.

128. Housner, G. W. Dynamic Pressure on Accelerated Fluid Containers, Bull. SSA, 47 : 15—35, 1957.

129. Jacobsen, Lydik S.; Ayre Robert S. Hydrodynamic Experiments with Rigid Cylindrical Tanks Subjected to Transient Motions. Bull. SSA 41 : 313—346, 1951.

130. Jacobsen, Lydik S. Impulsive Hydrodynamics of Fluid Inside a Cylindrical Tank and of Fluid Surrounding a Cylindrical Pier. Bull. SSA 39 : 189—204, 1949.

131. Amano Ryokichi; Azuma, Hisashi; Ishii, Yasumaru. Aseismic Design of Quay Walls in Japan. Paper No. 32, 17 pp. World Conference on Earthquake Engineering 1956.

132. Amano Ryokishi; Ishii Yasumaru; Tateishi, Tetsuro. Aseismic Design of Quay Walls in Japan. Monthly Report of

Transportation, Tech. Res. Instt. Tokyo. Vol. 6, No. 4, pp. 77—131, April 1956.

133. Am. Soc. Civil Engineers, San Francisco Sect. Subsidence and the Foundation Problem in San Francisco. Report of the Subsoil Committee, 107 pp., Sept. 1932.

134. Baumann, Paul. The Significance of Prestress in Embankment Dams. Proc., 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. VII, pp. 92—99, Rotterdam, 1948.

135. Branner, J. C. Impressions Regarding the Relations of Surface Geology to Intensity in the Mendoza, Valparaiso, Kingston and San Francisco Earthquakes. Bull. SSA, 1 : 38—43, 1911.

136. Casagrande, Arthur. Characteristics of Cohesionless Soils Affecting the Stability of Slopes and Earth Fills. Originally published in Jour., Boston Soc. Civil Engineers, Jan. 1936; Reprinted in 1940 in the Society's Volume Contributions to Soil Mechanics, 1925—1940, pp. 257—276.

137. Casagrande, Arthur. Classification and Identification of Soils. Trans. ASCE, 113 : 901—991, 1948.

138. Casagrande A.; Shannon, W. L. Research on Stress-Deformation and Strength Characteristics of Soils and Soft Rocks Under Transient Loading. Publ., Graduate School of Engineering, Harvard Univ., No. 447, (Soil Mechanics Series No. 31), 132 pp., 1948.

139. Clough, Ray W.; Pirtz, David. Earthquake Resistance of Rock-Fill Dams. Proc. ASCE, paper 941, 26 pp., April 1956.

140. Crockett, James Henry Archbold; Hammond, Reginald Egbert Rolt. Reduction of Ground Vibrations into Structures. Institution of Civil Engineers (London), Structural paper No. 18, 40 pp., 1947.

141. Drucker, M. A. Embedment of Poles, Sheeting, and Anchor Piles. (A Mathematical Treatment of Acting Forces and the Resistance of the Soil to Movement) Civil Engineering, 4 : 622—626, 1934. This method was used in designing the earthquake resistant steel pier at Puerto Armuelles, Panama, described in Civil Engineering, 7 : 187—191, 1937.

142. Duke, C. Martin. Bibliography on Effects of Soil Conditions on Earthquake Damags. Deptt. of Engg. Univ. of Calif., Los Angeles 24, California.

143. Esmiol, Elbert E. Seismic Stability of Earth Dams. U.S. Bureau of Reclamation Tech. Memo. No. 641, 36 pp., Denver, Color, April 1951.

144. Ewing, W. Maurice; Jardetzky, Wenceslas S.; Press, Frank. Elastic Waves in Layered Media. 380 pp., McGraw-Hill, 1957.

145. Forbes, Hyde. Landslide Investigation and Correction. Trans. ASCE, 112 : 377—442, 1947.

146. Glover, R. E.; Cornwell, F. E. Stability of Granular Materials Trans. ASCE, 108 : 46—82, 1943.

147. Gutenberg, B. Effects of Ground on Shaking in Earthquakes. Trans. AGU, 37 : 757—760, 1956.

148. Gotenberg, B. Effects of Ground on Earthquake Motion. Bull. SSA, 47 : 221—250, 1957.

149. Hatanaka, Motohiro. Fundamental Considerations on the Earthquake Resistant Properties of the Earth Dam. Bull. No. 11, 36 pp., Disaster Prevention Research Institute, Kyoto Univ., Kyoto, Japan, Dec. 1955.

150. Hennes, R. G. Research into the National Frenquency of Soils (Paper presented at Seattle, Washington, March 1954, before the Seismological Society of America; not published).

151. Rayleigh, John, William Strutt (Lord Rayleigh). The Theory of sound. 2 Cols. originally published 1877-78, 2nd ed., 1894—96. Vol. 1, 480 pp., Vol. II. 504 pp.

152. Rosenblueth, Emilio. Characteristics of Earthquakes.

153. World Conference on Earthquake Engineering, held at Berkeley, Calif, June 1956, under the Joint Sponsorship of the Earthquake Engineering Research Institute and Deptt. of Engineering, Univ. of Calif.

154. LeConte, Joseph N. The Stresses in a Free Prismatic Rod under a Single Force Normal to its Axis. Trans, ASCE, 91 : 968—990 (1927).

155. LeConte, Joseph N. Younger, John E. Stresses in a Vertical Elastic Rod When Subjected to a Harmonic Motion of One End. Bull. SSA, 22 : 1—37 (1932).

156. Looney, C.T.G. Steady-state Forced Vibration of Continuous Frames. Trans, ASCE, 118 : 794—828, 1953.

157. Martel R. R. The Dynamic Behavious of Some Simple Bents Subjected to Established Simple Harmonic Motion. Proc. World Engineering Congress, Tokyo, VIII, 213—222 (1929).

158. Mclean, R. S.; Moore, W. W. Computation of the Vibration Period of Steel Tank Towers. Bull. SSA, 26 : 63—67 (1936).

159. Misicu, M. On the Determination of Natural Periods of Vibration of Structures (In Rumanian, with French and Russian Abstracts). VI, Nos. 3-4, 447—455, (1955).

160. Misicu, M. Dynamic Stability of Rigid Structures on Columns or piling (in Rumanian, with French and Russian abstracts). Studii si Cercetari de Mecanics si Metalurgie, III : Nos. 3-4, 255—271, (1952).

161. Naito, Tachu, Earthquake-proof Construction. Bull. SSA, 17 : 57—94 (1927).

162. Moran, Donald F. Cheney, James A. Earthquake Response of Elevated Tank and Vessels. Proc. ASCE, Paper 1563, 14 pp. March, 1958.

163. Nazarov, A. G. Dispursion of Energy During Elastic Vibrations, in Russian) Reports Acad. of Sci., Armenian S.S.R., XVI : No. 3, 77—86, 1953.

164. Nazarov, A. G. Method of computation of Energy dispursion during Elastic Vibrations (in Russian) reports, Acad. of Sci., Armarian, SSR XVI : No. 5, 137—140, 1953.

165. Nazarov, A. G. Simplified method of computation of Seismic Resistance of Structures (in Russian) Bull. Acad. of Sci. Armenian S.S.R., IX : No. 10, 55—63 (1956).

166. Nazarov A. G. Studies of Internal Friction of Elastic Systems (in Russian) Armenian S.S.R., Acad. of Sci.

167. Newmann, Frank. Oscillator Responses to Earthquake motions. Oct. 1958, The trend in Engineer (quarterly, engineering Experiment Station, Univ. of Washington, Seattle).

168. Newmark N. M.; Veletsos, A. S. A Simple Approximation for the Natural Frequencies of Partly Restrained Bars. 6 pp., Tech. Rep. to ONR, Univ. of Illinois, Civil Engineering Studies, Structural Res. Series, No. 25, Urbana, Illinois, April, 1952.

169. Pan, Chia-Cheng. Vibration of Continuous Girders on Elastic Piers. Acta Geophsics Sinica, V : 153—176 (1956) (in Chinese).

170. Pan. S. L.; Goodman, L. E.; Newmark N. M. Influence of Ductility on the Response of Simple Structures to Earthquake motions, Tech. Rep. to ONR, Dept. Civil Engineering, Univ. of Illinois, Urbana, Illinois, 89 pp. 1951.

171. Priolo, Domenico, Dynamics of Antiseismic Construction. Giornale del Genio Civile, Roma (1951), 63, pp.

172. Rosenblueth, Emilio. Some Applications of Probability Theory in Aseismic Designs. Paper No. 8, 18 pp., World Conference on Earthquake Engineering, 1956.

173. Rosenblueth Emilio. Aseismic Designs of Simple Structures. Ediciones, ICA, B. 10 July, 1952.

174. Rosenblueth Amilio. Aseismic Design of Elastic Structures. Ediciones, ICA, B, 13, 31 pp., 15 Oct. 1952.

175. Rosenblueth, Emilio. Earthquake Proof Structures Founded on Firm Grounds. Ediciones, ICA, B. 17, 34 pp., 1 Nov. 1954.

176. Sutherlend J. G.; Goodman L. E. Vibrations of Prismatic Bars Including Rotary Inertia and Shear Corrections. 24 pp., Tech. Rep. to ONR, Dept. of Civil Engineering, Univ. of Illinois, Urbana, Illinois, April 1951.

177. Tanabashi, Ryo. Studies on the Non-linear Vibrations of Structures Subjected to Destructive Earthquakes. paper No. 6, World Conference on Earthquake Engineering, 1956.

178. Veletsos A. S.; Newmark N. M. A Simple Approximation for the Fundamental Frequencies of Two-span and Three-

span Continuous Beams, Tech. Rep. to ONR, Univ. of Illinois, Civil Engineering Studies, Structural Research Series. No. 66, Urbana, Illinois, Feb. 1954.

179. Veletsos A. S.; Newmark N. M. A Method for Calculating the Natural Frequencies of Continuous Beams, Frams and Certain types of plates. 212 pp., Tech. Rep. to ONR, Univ. of Illinois, Civil Engineering Studies, Structural Engg. Res. Series No. 58, Urbana Illinois, July 1953.

180. Veletsos A. S.; Newmark N. M. Natural Frequencies of Continuous Flexural Members. Trans. ASCE, 122: 249—285 (1957).

181. Wand, Kuang-Yuan. Methods of Determining Seismic Forces on Structures (in Chinese) Jour. of Civil Engineering V : 107—122 (1958).

182. Wang, Kuang-Yuan. Vertical Vibration of Tall Structures due to Seismic Effects. Chinese Jour. of Civil Engg. V : 140—149, (1958) (in Chinese).

183. Wilbur, John B. The Action of Impulsive Loads on Elastic Structures. Jour. Boston Soc. of Civil Engineers. XXXIII : 163—173 (1946).

184. Williams, Harry A. Dynamic Distortions in Structures Subjected to Sudden Earth Shock. Trans. ASCE, 102 : 838—868, 1937.

185. Mononobe, N. New Method of Improving the Seismic Stability of Reinforced Concrete Dams. Bull. FRI, Tokyo, 5 : 123—141, 1928.

186. Mononobe, N. Earthquake-proof, Construction of Masonry Dams. Proc. World Engineering Congress, Tokyo, IX : 275—294, 1929.

189. Priscu, R. Effects of Earthquakes on Gravity Dams and on Arch Dams. In Rumanian with abstracts in Russian and French; Studii si Ceretari de Mecanica si Metalurgie, Vol. II, 1951.

190. Priscu, R. Effects of Earthquakes on Gravity dams and on Arch Dams. In Rumanian with abstracts in Russian and French; Studii si Ceretari de Mecanica si Metalurgie, Vol. III, No. 3-4, 1952.

191. Senda, Kanae; Nakagawa, Kenji. On the Vibration of an Elevated Water Tank. Technology Reports of the Osaka Univ., Faculty of Engineering; Part I (by Senda & Nakagawa). Rep. 117, Vol. 4, pp. 247—264, Oct. 1954. Part II and III (by Nakagawa only), rep. 170 — 193. Vol. 5 pp. 317—336, Oct. 1955, and Vol. 6, pp. 53—62, March 1956.

192. Tishchenko, V. G.; Kharin, D. A. Vibrations of Hydraulic Structures, (in Russian). See Armenian S.S.R. Acad. of Sci., Sect. 7.1.

193. Tsien, Ling-hsi. Approximate Determination of Natural Frequencies for Arch Dam. Chinese Jour. of Civil Engineering, V : 123—131, 1958. (in Chinese).

194. Tsien, Ling-hsi.; Ch'iu, Tai-hung. Determination of Natural Frequency of Dam when full by Method of Electrical Analogy. Chinese Jour. of Civil Engineering, V : 132—139, 1958 (in Chinese).

195. Von Bergen, Harold E.; Nelidov, Ivan M. Earthquake Stresses in an Arch Dam. Trans. ASCE. 104 : 133—149, 1939.

196. Hennes, R. G. Highway Research Board. Landslides and Engg. Practice. Highway Res. Board Special Report 29, 232 pp., 1958.

197. Hodgson, Earnest A. The Marine Clays of Eastern Canada and Their Relation to Earthquake Hazards. Jour. Royal Astronomical Soc. of Canada, XXI : 257—264, 1927.

198. Iida, Kumibi. On the Elastic Properties of Soil, Particularly in Relation to its Water Content. Bull. ERI, Tokyo 18; : 675—691, 1940.

199. Ishimoto, Mishio; Iida, Kumizi. Determination of Elastic Constants of Soils by means of Vibration Method. Bull. ERI, Tokyo, Part I, Young's Modulus, 14 : 632—657, 1936. Part II, Modulus of Rigidity and Poissons's Ratio, 15 : 67—86, 1937.

200. Kanai, Kiyoshi, A Shord Note on the Relation of Amplitude period of Earthquake Motion. Bull. ERI, Tokyo, XXXII : 211—213, 1954.

201. Kanai, Kiyoshi. Semi-empirical Formula for the Seismic Characteristics of the Ground. Bull. Tokyo, XXXV : 309—324. 1957.

202. Kanai, Kiyoshi. The Requisite conditions for the Predominant vibration of ground. Bull. ERI, Tokyo, XXXV : 457—471, (1957).

203. Kanai. Kiyoshi; Suzuki, Masazi. Analytical Results of the Acceleration Seismograms obtained at Tokyo and Yokohama. Bull, ERI, Tokyo, XXXII : 189—197 (1954).

204. Kanai, Kiyoshi : Takahasi, Ryutaro; Kawasumi, Hirosi. Seismic Characteristics of Ground. Paper No. 31, 16 pp. World Conference on Earthquake Engineering (1956).

205. Kulik, J. Foundations of Earthquake-Resisting Structures.

206. Martel, R. R. Effect of Foundations on Earthquake Motion Civil Engineer, 10 : 7—10, (1940).

207. Minakami, Takeshi; Sakuma, Shuzo. The Earthquake-Motions on Various Formations of the Earth's Surface. Bull. ERI, Tokyo, 26; 61—71 (1948).

208. Miyabe, Naomi. A Study of Landslides. Bull. DRI, Tokyo, 13 : 85—113 (1934).

209. Mononobe, Nagaho; Matsuo, Haruo. On the Determination of Earth Pressure During Earthquake. Proc., World Engineering Congress, Tokyo (1929), IX : 171—186 (1929).

210. Mononobe, Nagaho; Matsuo, Haruo. Experimental Investigations of Lateral Earth Pressure during Earthquakes. Bull. ERI, Tokyo, 10 : 884—902, 1932.

211. Okamoto, Shunzo. Bearing Capacity of Sandy Soil and Lateral Earth Pressure during Earthquakes, Paper No. 27, 16 pp., World Conference on Earthquake Engineering, 1956.

212. Omote, Syun'itiro; Komaki, Shauzow; Kobayashi, Naoyoshi. Earthquake Observations in Kawasaki and Turumi Areas and the Seismic Qualities of the Ground. Bull. ERI, Tokyo, Part I : XXXIV : 335—364 (1956), Part II : XXXV : 613—619 (1957).

213. Rosenblueth, Emilio, Theory of Aseismic Design of Soft Strata. Ediciones ICA, B, 14 (15 Aug. 1953).

214. Sharpe, C. F. Steward, Landslides and Related Phenomena 137 pp., Columbia Univ. Press (1937).

215. Subsoil Research Team, Earthquake Research Institute, Tokyo. Investigation into Seismic Characteristics of Subsoils in Tokyo. Bull. ERI, Tokyo, XXXIII : 471—545 (1955).

216. Taylor, Donald W. Fundamentals of Soil Mechanics. 700 pp., Wiley (1948).

217. Terzaghi, Karl. Influence of Geological Factors on the Engineering Properties of Sediments. Economic Geology, Fiftieth Anniversary Vol., pp. 557—618 (1955).

218. Terzaghi, Karl. Mechanism of Landslides. pp. 83—123, Geol. Soc. Am. Berkey, Vol. (1950).

219. Trask, Parker D., Editor, Applied Sedimentation.

220. Tschebotarioff, Gregory P. Soil Mechanics, Foundations, and Earth Structures. 655 pp., McGraw-Hill, 1951.

221. Bernhard, Rudolph K. Dynamic Test by means of Induced Vibrations. Proc. ASTM, Vol. 37, Part II, pp. 634—649, 1937.

222. Crandell, F. J. Ground Vibrations due to Blasting and its Effects Upon Structures. Jour., Boston Soc. Civil Engineers, 36 : 224—245, 1949.

223. Engg. New Record Editorial Staff. Vibration Effects on Foundations and Masonry Structures. Eng. News Rec., 104 : 900-901, May 29, 1930.

224. Guha, S. K.; Ram G.; Rao G. B. Vibration Characteristics of Khadakwasla Dam (Poona, India). Irrigation & Power (the Jour. of the Central Board of Irrigation & Power, Simla, India), Vol. 11, No. 2, pp. 239—247, April 1954.

225. Hudson, D. E.; Alford, J. L.; Housner, G. W. Response of a Structure to an Explosive-Generated Ground Shock. Tech. Rep. to ONR, Eq. Res. Lab., Calif. Inst. Tech., Pasadena, 39 pp., 1952.

226. Hudson, D. E.; Alford, J. L.; Housner, G. W. Measured Response of a Structure to an Explosive-Generated Ground shock. Bull. SSA, 44 : 513—527, 1954.

227. Jarapetyan, B. K. Results of Seimometric Observations during Large Blasts. (in Russian) Bull., Acad. of Sci., Armenian S.S.R., X : No. 3, 21—34, 1957.

228. Leet, L. Don. Ground Vibrations near Dynamite Blasts. Bull. SSA, 29 : 487—496 (1939).

229. Murphy, V. A. New Zealand Earthquake Problem in relation to Engineering Structures. The New Zealand Institution of Engineers, Proc., 32 : 302—335, 1946.

230. Neumann, Frank. Damaging Earthquake and Blast Vibration. The Trend in Engineering (Publ. quarterly by the Engg. Experiment Station, Univ. of Washington, Seattle, Washington), Jan. 1958, pp. 5—10, 24—28.

231. Patterson, Wm. D. Determination of Ground Periods. Bull. SSA, 30 : 129—138, 1940.

232. Robinson, Quentin R. Vibration Machines at the David W. Taylor Model Basin. Dept. of the Navy, Taylor Model Basin Rep. No. 821, 63 pp., 1952.

233. Suyehiro, Kyoji. Engineering Seismology, Notes on American Lectures. 110 pp., Proc. ASCE, Vol. 58, No. 4, May 1932.

234. Tanabashi, Ryo; Ishizaki, Hatsuo. Earthquake Damages and Elastic Properties of the Ground. Bull. No. 4, pp. 1—70, Disaster Prev. Res. Instt., Kyoto Univ., Kyoto, Japan (May 1953).

235. U.S. Experimental Model Basin. Translation No. 51. Theory and Practice of Vibration Testing Machines by Wilhelm Spath, originally published by Julius Springer, 1934.

236. Abe, Mikishi. Tests of Reinforced Concrete Frames under Repeated Horizontal Loads and Measurement of their periods of Free Vibrations. Proc., World Engineering Congress, Tokyo, VIII : 501—559, 1929.

237. Barkeley, E. C.; Wainwright, L. Computors; Their Operation and Application. 366 pp., Reinhold Publ. Corp., 1956.

238. Bierer, R. W.; Parker, T. B. An Experimental Investigation on the Resistance of Mansonry to Vibration with Special Reference to the Effect of Earthquake Waves. Bull SSA, 1 : 107-108, 1911.

239. Bykhovskii, V. A. Solution of Some Problems of Seismic Construction by Experimental Study of Models. (in Russian). See Armenian S.S.R., Acad. of Sci.

240. Caughey, T. K.; Husdon, D. E. A Response Spectrum Analyzer for Transient Loading Studies, Proc. Soc. for experimental Stress analysis, Vol. XIII, No. 1, pp. 199—206, 1955.

241. Gaul, Roy D. Model Study of a Dynamically Laterally Loade Pile. Proc. ASCE, Paper 1535, 33 pp., Feb. 1958.

242. Freberg, C. R.; Kemler, Emory N. Elements of Mechanical Vibration. 227 pp., 2nd ed., Wiley (1949). Chap. X. pp. 198—216.

243. Hetenyi, M.; Editor-in-Chief. Handbook of Experimental Stress Analysis. 1077 pp., 820 illus., Wiley (1950).

244. Housner, G. W.; Hudson D. E. Applied Mechanics-Dynamics. 255 pp., Van Nostrand (1950). Dimensional Analysis is treated in Chap. 1, pp. 9—18.

245. Housner G. W.; Martel, R. R.; Alford J. L. Spectrum Analysis of Strong-motion Earthquakes. Bull. SSA, 43 : 97—119, 1953.

246. Housner, G. W.; McCann, G. D. The Analysis of Strong-motion Earthquake Records with the Electric Analog Computer. Bull. SSA, 39 : 47—56, 1949.

247. Hudson D. E.; Lurie, H. High Speed Computing Methods for Shock and Vibration Problems. pp. 165—188 of the Vol. entitled Shock and Vibration Instrumentation, publ. by Am. Soc. of Mechanical Engineers, (1956).

248. Jacobsen, Lydik S. Vibration Research at Stanford Univ. Bull. SSA, 19 : 1—27, 1929.

249. Jacobsen, Lydik S. Dynamics of Structural Models in U.S.A.; paper appearing in I Modelli nella Tecnica, Vol. I, pp. 12—34, 1956.

250. Karapetyan, B. K. Experimental Determination of Internal friction of Stone Masonry (in Russian). Bull., Acad. of Sci., Armenian S.S.R., IX : No. 8, 70—83, 1956.

251. Karapetyan, B. K. Method of Determining Parameters for Internal Friction of Materials During Free Vibrations (in Russian). Bull., Acad. of Sci. Armenian S.S.R., VII : No. 2, 19—29, 1954.

252. Korn, Granino A.; Korn, Theresa M. Electronic Analog Computers. 378 pp., McGraw-Hill (1952).

253. Langhaar, Henry L. Dimensional Analysis and Theory of models. 166 pp., Wiley (1951).

254. Lee, George Hamor. An Introduction to experimental stress analysis. 319 pp., Wiley (1950).

255. Minami, J. K. Sell type Foundation Construction for Earthquake Resistance.

256. Mononobe, Nagaho; Matsuo, Haruo. On the Determination of Earth Pressure During Earthquake. Proc., World Engineering Congress, Tokyo (1929), IX : 171—186 (1929).

257. Mononobe, Nagaho; Matsuo, Haruo. Experimental Investigation of Lateral Earth Pressure During Earthquakes. Bull. ERI Tokyo, 10 : 884—902 (1932).

258. Morris, Brooks T. A Laboratory Model Study of the Behaviour of Liquid-filled Cylindrical Tank in Earthquakes. Thesis, Dept. of Civil Engineering, Standford Univ., June 1938.

259. Murphy, Glenn. Similitude in Engineering. 302 pp., The Ronald Press (1950).

260. Murphy, M. J.; Bycroft, G. N. Response of a Non-linear Oscillator to an Earthquake. Bull SSA, 46 : 57—65 (1956).

261. Nazarov A. G. Mechanical Similitude of Solid Bodies and its Applications in the Investigation of Seismic Resistance of Structures (in Russian) Bull. Acad. of Science Armenian S.S.R., X : No. 5, 15—28, 1957 and X : No. 6, 3—16, 1957.

262. Nazarov, A. G. Phenomenon of Hysteresis During Elastic Vibration (in Russian) Bull. Acad. of Sci., Armenian S.S.R., VI : No. 4, 21—33, 1953.

263. Oberti, Guido. Development of Aseismic Design and Construction in Italy by Means of Research on Large Model Test. Paper No. 25, pp. 12, World Conference on Earthquake Engineering, 1956.

264. Oberti, Guido. Experimentation on Structural Models in Italy, Vol. 1, pp. 35—55, published by Acad. Nazionale dei lincei, Rome, 1906.

265. Portnob, M. A. Concerning Shaking Table Experiments (in Russian). Proc. of Conference on Seismic Construction, 271 pp., 1956.

266. Ramirez, John R. Failure of Model Sand Dams Subjected to Simulated Earthquakes. Thesis Dept. of Civil Engg. Stanford Univ., 63 pp., June 1941.

267. Rogers, F. J. Experiments with a Shaking Machine. Calif. State Earthquake Investigation Commission. Vol. I, Part 2, pp. 326—335.

268. Ruge, Arthur C. The Determination Earthquake Stresses in Elastic Structure by Means of Models. Bull. SSA, 24; 169—230, 1934.

269. Ruge, Arthur C. A Machine for Reproducing Earthquake Motions Direct from a Shadowgraph of the Earthquake. Bull. SSA, 26 : 201—205, 1936.

270. Soroka, Walter W. Analog Methods in Computation and simulation, 390 pp., McGraw-Hill (1954).

271. Soroka, Walter W. Recent Advances in Analogic models in Applied Mechanics. Paper appearing in I Modelli Nella Tecnica, Vol. II, pp. 185—201, 1956.

272. Stelson, Thomas E.; Mavis, Frederic T. Virtual Mass and Acceleration in Fluids. Trans. ASCE, 122 : 518—530, 1957.

273. Stibitz, George R.; Larrivee, Jules A. Mathematics and Computer, 228 pp., McGraw-Hill (1957).

274. Timoshenko, S.; Young D. H. Advanced Dynamics, 400 pp., McGraw-Hill (1948).

275. Tingley, S. F. The Application of Stress Models to Specific Structural Problems. Proc. Soc. for Experimental Stress Analysis, Vol. 3, No. 1, pp. 71—80, 1945.

276. Bourgin, A. The Design of Dams. Translated from the French by Frank F. Ferguson, 334 pp., Pitman London, 1953.

277. Creager, William P.; Justin J. D.; Hinds, J. Engineering for Dams. Three Vols., Wiley, 1945.

278. Engineering News Record. Flexible Core for Dam in Earthquake Country. Engg. New Rec. 125 : 279, Aug. 29, 1940.

279. Foster, H. A. Formulas Indicate Earthquake Forces in Design of Ring-girder Supported Pipes. Civil Engg. 19 : 45-46, 1949.

280. Foster, H. A. Slopping Surge Tank Designed to Resist Earthquake Forces. Engg. News Record. 141 : 93—95, Nov. 11, 1948.

281. Galloway, J. D. A Design of Rock-fill Dams. Trans. ASCE, 104 : 1—92, 1939.

282. Hammond, J. J. Basic Design Criteria for Concrete Gravity and Arch Dams. Proc. ACI, 50 : 657—668, 1954.

283. Hazen, A. Safety in Dam Construction. Proc. World Engg. Cong., Tokyo, IX : 251—274, 1929.

284. Heiland, C. A. Geophysical Investigations Concerning the Seismic Resistance of Earthdams. Trans. Am. Inst. Mining and Metallurgical Engg. 138 : 350—377, 1940.

285. Horie, K. Earthquake Disaster and the Reconstruction of Water Works in Yokohama. Proc. World Engineering Congress, Tokyo, XII : 99—143, 1929.

286. Houk, Ivan E.; Keener, Kenneth E. Masonry Dams—Basic Design Assumptions. Trans. ASCE, 106 : 1115—1130, 1941.

287. Loudorbael, George, D. Faults and Engineering Geology. This paper will be found on pp. 125—150 of the Geological Soc. of Am., Berkey Vol., Publ. 1950.

288. Middlebrookes, T. A. Earth-Dam Practice in the United States. Am. Soc. Civil Engineers, Centennial Trans. Vol. CT, pp. 697—722, 1953.

289. Morris, Samuel B. Discussion of paper by J. D. Galloway on The Design of Rock-Fill Dams. Trans. ASCE, 104 : 35—38, 1939.

290. Morris, S. B.; Pearce, C. E. Design of Gravity Dam in San Gabriel Canyon to Resist Earthquokes. Bull. SSA, 19 : 143—155, 1929.

291. Parsons, H. de B. Hydrostatic Uplift under Masonry Dams. Proc. World Engineering Congress, Tokyo, IX : 295—314, 1929.

292. Pracy, Geo. W. Disaster Preparedness Plans for Water Works. Jour. Am. Water Works Assoc., 22 : 1200—1214, 1930.

293. Semenza, Carlo. Dams in Italy Constructed for Earthquake Resistance. Paper No. 37, 5 pp., World Conference on Earthquake Engineering, 1956.

294. Tennessee Valley Authority. Civil and Structural Design. TVA Tech. Rep. No. 24, Vol. 1, 439 pp., 1952.

295. Tennessee Valley Authority. The Kentucky Project. TVA Tech. Rep. No. 13, 877 pp., 1951.

296. U.S. Bureau of Reclamation. Boulder Canyon Project—Final Reports, Part V : Technical Investigations; Bull. 4 : Stress Studies for Boulder Dam. 286 pp., Denver, Colorado, 1939.

297. Beles, Aurel A. The Earthquake of Nov. 10, 1940 and its effects on structures. 20 pp., Session Reports of the Rumanian Acad. of Sci., Vol. V, No. 3, 1941.

298. Binder, R. W. Engineering Aspects of the 1933 Long Beach Earthquake. Proc., Symposium on Earthquake and Blast Effects on Structures, pp. 186—211, 1952.

299. Bruggen, Juan. Contribution to the Seismic Geology of Chile. 132 pp., Imprenta Universitaria, Santiago de Chile, 1944.

300. Central Board of Geophysics, India. A compilation of Papers on the Assam Earthquake of Aug. 15, 1950. Compiled by M. B. Ramachandra Rao, Govt. of India, Central Board of Geophysics, Calcutta, Publ. No. 1, 112 pp., 1953.

301. Far East Command, General Headquarters, Office of the Engineer. The Fukui Earthquake, Hokuriku Region, Japan, 28 June, 1948. Vol. II : Engineering, 205 pp., maps & illustr.

302. Galanopoulos, A. G. The Koroni (Messinia) Earthquake of Oct. 6, 1947. Bull. SSA, 39 : 33—40, 1949.

303. Hiller, Wilhelm. Why "Regulations for Structures in Earthquake Zones" in Germany. Paper No. 18, 4 pp., World Conference on Earthquake Engineering, 1956.

304. Japanese Soc. of Engineers. Rep. on the Investigation of Shock of the Great Earthquake in Northeastern Japan in 1823 (In Japanese).

305. Karapetyan, B. K.; Shakhsuvaryan, L. V. Results of Engineering Observations of the Burakanskogo Earthquake (in Russian). Bull., Acad. of Sci., Armenian S.S.R., VII : No. 5, 91—95, 1954.

306. Lendrum, H. L. Effect of Quetta Earthquake on Various types of Construction. Royal Engineers Jour. (Instt. of Royal Engineers), 50 : 327—343, 1936.

307. Miller, Alfred L. Earthquake Lessons from the Pacific North-west. Proc., Symposium on Earthquake and Blast Effects on Structures, pp. 212—223, 1952.

308. Neumann, Frank. Earthquake Intensity and Related Ground Motion, 77 pp., Univ. of Washington Press, Seattle, 1954.

309. Peruzyan, S. A. Influence of Local Geology upon the Magnitude of Earthquake Intensity during Gegechkorskogo Earthquake of 1957 (in Russian). Bull., Acad. of Sci., Armenian S.S.R. X : No. 6, 67—74, 1957.

310. Schlocker, J.; Radbruch, Dorothy H. Arvin-Tehachapi Earthquake Structural Damage as related to Geology. Part III, Paper No. 1, pp. 213—220, Calif. Div. Mines, Bull., 171, 1955.

311. Sparks, Neil R. The Eureka Earthquake of June 6, 1932. Bull. SSA, 26 : 13—27, 1936.

312. Steinbrugge, Karl V. : Bush, Vincent R. Earthquake Damage can be Controlled. Consulting Engineer, May 1957. pp. 74—79.

313. Steinbrugge, Karl V.; Moran, Donald F. An Engineering Study of the Southern Calif. Earthquake of July 21. 1952 and its after-shocks. Bull. SSA, 44 : 201—462, 1954.

314. Steinbrugge, Karl V.; Moran, Donald F. Damage Caused by the Earthquake of July 6, and Aug. 23, 1954. (Fallen-Stillwater, Nevada). Bull. SSA, 46 : 15—33, 1956.

315. Steinbrugge, Karl V.; Moran, Donald F. An Engineering Study of Euraka, Calif. Earthquake of Dec. 21, 1954. Bull. SSA, 47 : 129—153, 1957.

316. Steinbrugge, Karl V.; Moran, Donald F. The Dixie Valley—Fairview Peak, Nevada, Earthquakes of Dec. 16, 1954. Engineering Aspects Bull. SSA, 47 : 335—348, 1957.

317. Tanabashi, Ryo; Ishizaki Hatsuo. Earthquake Damages and Elastic Properties of the Ground. Bull. No. 4, pp. 1—70, Disaster Prevention Research Instt., Kyoto Univ., Kyoto, Japan, May 1953.

318. Willis, Bailey. Earthquake Conditions in Chile. Carnegie Institution of Wash. Publ. No. 382, 178 pp. 1929.

319. Wood, Harry O. Distribution of Apparent Intensity in San Francisco (Calif. Earthquake of April 18, 1906).

320. Wood, Harry O. "Apparent" Intensity and Surface Geology. Chap. 7, pp. 67—82 of Physics of the Earth, Vol. VI, Seismology.

321. Wood, Harry O. Preliminary Report on the Long Beach Earthquake of March 10, 1933. Bull. SSA, 23 : 43—56, 1933.

322. Dewell, H. D.; Huber, W. L.; Jacobsen, L. S. Discussion of Paper Earthquakes and Structures by Hoskins and Galloway.

323. Far East Command. The Fukui Earthquake, Hokuriku Region, Japan, 28 June 1948. Vol. II, Engg., 205 pp., Maps & Illustr.

324. Moran, Donald F.; Cheney James A. Earthquake Response of Elevated Tanks and Vessels. Proc. ASCE, Paper 1563, 14 pp., March, 1958.

325. Robinson, I. R.; Benjamin, H. L. Effects of Earthquakes on Electrical Supply Systems. (28 pp. & map) (Paper read at the Annual General Meeting of the New Zealand Soc. of Civil Engineers, held at Christ Church, Feb. 21 to 25, 1933).

326. U.S. Geological Survey Bull. 324. The San Francisco Earthquake and Fire of April 18, 1906 and their Effects on Structural Materials. 170 pp., 57 pl. (1907).

327. Deem, J. S. L. Remarks on the Damage to Water Supply and Sanitary Systems at Napier, New Zealand in the earthquake of Feb. 3, 1931. Proc., Institution of Civil Engineers, 236 : 402—407, 1934.

328. Louderback, George D. Characteristics of Active Faults in the Central Coast Ranges of California, with Application to the Safety of Dams. Bull. SSA, 27 : 1—27, 1937.

329. O'Shaughnessy, M. M. Sheffield and Gem Lake Dam Failures. Eng. News Rec., 95 : 194— (1925).

330. Schussler, Hermann. The Water Supply of San Francisco, Calif. Before During and After the Earthquake of April, 18, 1906, and Subsequent Conflagration. July, 1906.

331. Abbett, Robert W. American Civil Engg. Practice. Vols. 1 & 2, Wiley 1956, Vol. 3, Wiley 1957.

332. Andersen P. Substructure Analysis and Design. The Ronald Press, 1956.

333. Beedle, L. S. Plastic Strength of Steel Frames. Trans. ASCE, 122 : 1139—1168, 1957.

334. Bleich, S. F. With the Collaboration of Lyle B. Ramsey. Buckling Strength of Metal Structures. (Engineering Soc. monographs) McGraw-Hill, 1952.

335. Chellis, R. D. Pile Foundations, theory, Design, Practice. 681 pp., McGraw-Hill, 1951.

336. Davis, H. E.; Troxell, G. E.; Wiskocil. Clement T. The Testing and Inspection of Engineering materials. 431 pp., McGraw-Hill, 1955.

337. Komendant, A. E. Prestressed Concrete Structures. 261 pp., McGraw-Hill, 1952.

338. Krynine, D. P.; Judd, W. R. Principles of Engg. Geology and Geotechnis. 730 pp., McGraw-Hill, 1957.

339. Krynine, D. P. Soil Mechanics, its Principles and Structural Applications. 511 pp., 2nd Ed, McGraw-Hill, 1947.

340. Legget, R. F. Geology and Engineering. 650 pp., McGraw-Hill, 1939.

341. Lin, T. Y. Design of prestressed Concrete Structures, 456 pp., Wiley, 1955.

342. Neal, B. G. The Plastic Methods of Structural Analysis, 353 pp., Wiley, 1956.

343. Am. Soc. of Civil Engineers. Earthquake Committee Report, 1928.

344. Am. Soc. for Testing Materials. Symposium on Seismic and Shock Loading held at Los Angeles, Calif. Sept. 17, 1956. ASTM Special Tech. Publ. No. 209.

345. Armenian Soviet Socialist Republic, Acad. of Sci. Proc. of Conference on Seismic Consn., 271 pp., 1956.

346. Barrera, T. The Earthquake of Jan. 14, 1931. 40 pp., Institute de Geologia, Univ. Nacional de Mexico, 1932.

347. Beles, A. A. Earthquakes and Construction. Buletinul Soc Politecnice, Bucharest, LV : 1045—1094, 1171—1211, Oct. and Nov. .1941 (in Rumanian).

348. Binder, R. W. Trends in Earthquake Resistant Design. Paper Presented at San Francisco Regional Technical Meeting of American Iron and Steel Institute, 9 pp., Nov. 1954.

349. Brunnier, H. J. The Development of Aseismic Construction in the U.S. paper No. 26, World Conference on Earthquake Engg., 1956.

350. Ch'eng, Wen-siu. Earthquake Resistant Problems of Masonry Structures. Shinese Jour. of Civil Engg., V : 150—164; 1958 (in Chinese).

351. Clough, R. W. Use of Modern Computers in Structural Analysis. Proc. ASCE, Paper 1936, May 1958.

352. Derrick, C. J. Damage Potential of Earth-shocks. Proc. Annual Convention, Structural Engineers Assoc. of Calif, held at Coronado, Calif, 1954.

353. Derrick, C. J. Elements of Aseismic Design. Part I, physical and theoretical background. 74 pp., Publ. by the author, 1955.

354. Dewell, H. D. Progress of Earthquake Resistant Design. Civil Engineering 9 : 601—604, 1939.

355. Dunham, S. W. Foundations of Structures. 679 pp., McGraw-Hill, 1950.

356. Dye, E. R. Earthquake Resisting Strutures, Montana State College. Engg. Experiment Station, Bull. 1, 45 pp., Bozeman, Montana, Jan. 1936.

357. Edwards, H. H. Construction for Safety in Earthquakes. The Military Engineer, Vol. XLII : No. 286, pp. 94—97, March-April, 1950.

358. Golden Blat, E. E. & Bykhovskii, V. A. Editors. Construction in Seismic Regions, 172 pp., Federal Publ. on Structures and Architecture, Moscow, 1957 (in Russian).

359. Gumensky, D. B. Earthquakes and Earthquakes Resistant Design. Sect. 34, pp. 34—01 to 34—34, Vol. 3 of Am. Civil Engg. Practice.

360. Hiroi, I. Prevention of Damages to Engg. Structures Caused by Great Earthquakes. Proc. World Engg. Congress, Tokyo, IX : 25—40, 1929.

361. Housner, G. W. Current practice in the Design of Structures to Resist Earthquakes. Bull. SSA, 39 : 169—179. 1949.

362. Housner, G. W. The Design of Structures to Resist Earthquakes. Paper No. 5, 13 pp., World Conference on Earthquake Engineering, 1956.

363. Ibanez, V. J. Stability and Earthquake Resistant Features of Structural Designs. (Annals fo the Faculty of Physical Sciences and Mathematics, Univ. of Chile), Santiago. 1 : 108—120, (1944); 2 : 94—123, (1945); 3 : 64—70, (1946).

364. Proceedings of the Second World Conference on Earthquake Engineering—Tokyo and Kyoto, Japan. July 11—18, 1960.

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414. Neuman, F. A Lateral-force Formula Based on Seismological Concepts—The Trend in Engg., Vol. 12, No. 3, 1960. 07.

415. Tsui, Y. W. Aseismic design of Structure by rigidity criterion. Proc. Am. Soc. of C. E., Jor. of Structural Engineering, Vol. 85, No. ST5, 1959. 02.

416. Blumf, J. A. Structural Dynamics in Earthquake Resistant Design Proc. Am. Soc. of C. E., Jor. of Structural Division, Vol. 84, No. ST4, 1958. 07.

417. Roorkee University. Earthquake Engineering Seminar held at the University of Roorkee from 10th to 12th February, 1959. University of Roorkee, U.P., India, Seminar Series No. 1.

418. Earthquake Resistant Design of Civil Engineering. Structures—Earth Structures and Foundations in Japan—The Japan Society of C.E. 1960.

419. Howells, D. A. Structures to Withstand Earthquakes, Engineering, Vol. 187, No. 4853, 1959. 03. 13.

420. Penzien, J. Dynamic Response of Elesto-plastic Frams Proceedings. Am. Soc., C.E. Structural Division, Vol. 86, No. ST7, 1960.

421. Wen-XI, H. Analysis of the Structural Action of Multiple Arch Dam Under Lateral Earthquakes. Scientia Sinica., Vol. VIII, No. 8, 1959. 08.

422. R. N. Joshi. Structural Engineer, Director, S. B. Joshi & Co. (P.) Ltd., Bombay. Aseismic Construction in India, Earthquake Engineering Seminar, University of Roorkee, 1959.

423. P. C. Hazra, V. S. Krishnaswamy & J. Swami Nath. Geological Survey of India, Calcutta. Regional Geology and Seismicity of the Bhakra Dam Project, Earthquake Engineering Seminar, University of Roorkee, 1959.

424. V. S. Krishnaswamy, Senior Geologist, Geological Survey of India. Seismicity of the Beas-Sutlej Link Project, Earthquake Engineering Seminar, University of Roorkee, 1959.

425. V. S. Krishnaswamy, Senior Geologist, Geological Survey of India. Seismicity of the Konch Dam Project, Earthquake Engineering Seminar, University of Roorkee, 1959.

426. J. B. Auden, Ex-Sperintending Geologist, Geological Survey of India. Earthquakes in Relation to the Damodar Valley Project, Earthquake Engineering Seminar, University of Roorkee, 1959.

427. Dr. R. S. Mithal & L. S. Srivastava, University of Roorkee, Roorkee. Geotectonic Position and Earthquakes of Gango-Brahamputra Region, Earthquake Engineering Seminar, Roorkee, 1959.

428. P. C. Hazra & B. N. Raina, Geological Survey of India. Earthquake and some Related Engineering Problems. Earthquake Engineering Seminar, University of Roorkee, 1959.

429. Tadashi Hatano, Tadashi Takahashi & Hazime Tsutsumi, Technical Research Laboratory, Central Institute of Electric Power Industry, Tokyo. Dynamical Study of Tsukabaru Gravity Dam, Earthquake Engineering Seminar, University of Roorkee, 1959.

430. I. P. Kapila, Deputy Director, Bhakra, Design Directorate, New Delhi. Aseismic Design for Bhakra Dam. Earthquake Engineering Seminar, University of Roorkee, 1959.

431. Dr. S. K. Guha & G. V. Rao, Central Water & Power Research Station, Poona. Vibration Studies of Dams & Foundations. Earthquake Engineering Seminar, University of Roorkee, 1959.

432. C. D. Mitra, Superintending Engineer, Investigation and Project Circle, Patna. Seismic Consideration in Design of Dams. Earthquake Engineering Seminar, University of Roorkee, 1959.

*APPENDIX O*

## PROCEDURE FOR EARTHQUAKE RESISTANT DESIGN OF DAMS BEING FOLLOWED IN INDIA

### 1. Introductory

In India, the Himalayan and the sub-Himalayan regions, Indo-Gangetic plains, the State of Assam and Kutch and Kathiawar regions are subjected to earthquakes of varying degree from times immemorial. However, the records of the earthquakes and the damage caused in the recent past are only available. Some of the more important of these which have caused great havoc are the 1905 earthquake of Kangra, 1934 earthquake of Bihar, 1935 earthquake of Quetta and 1950 earthquake of Assam. The Assam earthquake of 1897, records of which are available in some detail, is recognized to be one of the worst earthquakes of history. The earthquake zoning map showing the regions in which earthquakes of different intensities are normally experienced, as prepared by ISI may be seen at appendix 'S'.

Recently few dams like Bhakra, Kosi, Rihand, Barapani and Beas have been completed or are already under construction in the seismic zones. Many more dams like Kopili, Ramganga, Barak, Tirap etc. are under investigation which would help to tap the great irrigation and power potential that exists in the rivers flowing through the seismically active belts of India. In order to take care of any potential damage due to earthquakes, there is a need for evaluation of an earthquake resistant design procedure for dams in India. Recently, a standard code of practice for seismic design of engineering structures has been prepared by Indian Standards Institution—No. IS: 1893-1962. So far the practice of examining the stability of the dam structure for the earthquake conditions by model tests has not been developed in India. As a result, certain design criteria are being followed. These are described below.

### 2. General Design Criteria

As in other countries, seismic coefficient method of statical analysis is adopted for seismic design of dams in India. This design is underlined by considerations of economy. Due to the short duration and infrequent occurrences of earthquakes and due to the fact that failure of a structure is almost unavoidable in case of earthquake of magnitude high enough to cause a permanent rupture, the section designed for earthquake conditions is reasonably increased in size to accommodate forces likely to be set up during the apprehended earthquake. The period of vibration is generally taken as 1 second.

Usually, the designs are based on the most adverse combination of probable load conditions but these include only those loads having reasonable probability of simultaneous occurrence. Combinations of transient loads, each of which has only a remote probability of occurrence at any given time have normally no probability of simultaneous occurrence and are therefore not considered as a reasonable basis for design. For example, maximum earthquake should not be combined with maximum design flood. While checking the section of dam under earthquake conditions, relaxations in permissible stresses and factor of safety are allowed for the condition of maximum earthquake and the most adverse combination of other conditions. In fact, design of a non-tension section is not insisted upon and certain permissible tension is allowed to develop under these conditions. The direction of action of earthquake acceleration is taken normal to the axis of the dam in the case of gravity and earth dams.

**3. Seismic Coefficient**

The seismic coefficient, expressed as a ratio of earthquake acceleration to the gravity acceleration, to be allowed in the design of a dam is determined by considering the following factors:

(a) Type and life span of the dam.

(b) Type of foundation.

(c) The magnitude and frequency of occurrences of earthquakes in areas where the dam is to be constructed.

(d) Location of active faults and distances of the site of the structure from these.

Seismic coefficients used in some of the dams recently designed in India may be seen at appendix 'R'. The procedure adopted for the seismic design of masonry and earth dams is given below with a few examples.

**4. Masonry Dams**

In the design of dam, the earthquake force is generally considered to act perpendicular to the axis. In the design of masonry dams, the following earthquake forces are considered:

(a) *Effects of horizontal earthquake acceleration on the horizontal and vertical component of reservoir and tail water load.*—Due to the horizontal acceleration of the foundation and dam, there is an instantaneous hydro-dynamic pressure exerted against the dam in addition to the hydrostatic forces. The direction of this hydro-dynamic force is opposite to the direction of the earthquake acceleration. Based on the assumption that water is incompressible, the pressure at any depth y below the reservoir surface is expressed by the following equation:

$$P_e = C\alpha_h Wh$$

where

$P_e$=hydrodynamic pressur at depth y,

W=unit weight of water,

h=maximum depth of reservoir,

$\alpha_h$=horizontal seismic coefficient, and

C=Coefficient which varies with shape of dam and depth.

(b) *Masonry inertia force due to horizontal earthquake acceleration.*—The horizontal inertia force of concrete or masonry weight due to horizontal earthquake acceleration acting at any horizontal section of the dam is the product of the weight above the section and the seismic coefficient, as expressed by the following equation :

$$V_i = \alpha_h W$$

where

$V_1$=Horizontal inertia force of concrete or masonry,

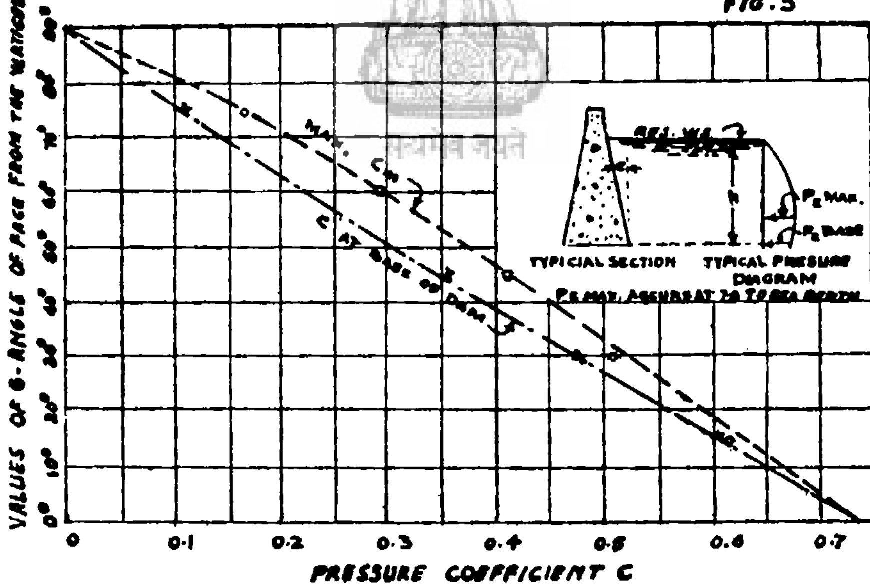
$W$=Dead load above the section under consideration, and

$\alpha_h$=Horizontal seismic coefficent.

This inertia force also acts in a direction opposite to the direction of the earthquake acceleration and it acts at the centre of the gravity of the mass above the section. It causes an overturning moment about a horizontal section, adding to that caused by the hydrodynamic force.

(c) *Effect of vertical earthquake acceleration.*—The effect of vertical acceleration is to change the unit weight of water, concrete and masonry. The upward acceleration increases the weight; while the downward acceleration decreases it. Vertical and horizontal forces and moments computed for the analysis without earthquake effects are therefore multiplied by the factor $(1 \pm \alpha)$ to include the effect of vertical acceleration.

FIG. 5




PRESSURE COEFFICIENTS FOR CONSTANT SLOPING FACES.

(d) *Effect of earthquake acceleration on uplift forces.*—Effect of earthquake acceleration on the uplift forces at any horizontal section is determined as a function of the hydrostatic pressure of reservoir and tailwater against the faces of the dam. During an earthquake, the water pressure is changed by the hydro-dynamic effect. However, the change is not considered effective in producing a corresponding increase or reduction in the uplift force. The duration of the earthquake is too short to permit the building up of pore pressures in the concrete and foundations. Therefore, normal uplift force is assumed to have such an intensity that at the line of drains, it exceeds the tailwater pressure by one-third the difference between the headwater and tailwater. The pressure gradient is extended to the headwater and tailwater in straight lines and the pressure is assumed to act over 100 per cent of the area.

(e) *Effect of earthquake acceleration on dead silt loads.*— Experimental and analytical methods both indicate that earthquake acceleration is only about one-half as effective in silt or soil masses as it is in water. This is mainly due to internal shear resistance of the silt. Since the unit weight of water is also approximately ½ that of silt, it is sufficient to determine the increase in silt pressure due to earthquake as if the water is extended to the base of the dam. This increase is then added to the static silt pressure.

In important and very high structures like Bhakra Dam, with the provision of keyed and grouted joints, study is made under trial load analysis considering transfer of some portion of internal loads under the extreme earthquake conditions to the abutments. Two examples of the dams recently designed are given below:

*Barapani*

Proposed Barapani Dam site is across Umiam river in Shillong, Assam State, is situated in a highly seismic zone. Geological structure of the area shows that entire Shillong Plateau has to be considered as unstable and susceptible to earthquakes. It may be pointed out that no less than 32 earthquakes of magnitude varying from 6.0 to 8.6 on Richter's scale have been experienced by this region between years 1906 to 1952. Out of these at least 11 shocks were having greater magnitude than 7.5.

In the design of this 236′ high concrete dam it was decided to adopt a factor of 0.225 g for horizontal acceleration and 0.1 g for vertical acceleration. The analysis of the blocks was carried out in the most unfavourable directions of the earthquake and a period of vibration of 1 second was assumed.

*Kosi Project*

This project lies in an active seismic zone. The investigations after 1934 Nepal-Bihar earthquake placed the area in intensity IX of the 1931 Mercalli modified scale (equivalent acceleration between 0.24 g and 0.46 g) the seismic coefficients, adopted for the design of barrage and the earthen embankment are as follows:

*Barrage*

| | | |
|---|---|---|
| Horizontal earthquake acceleration | .. | 0.15 g |
| Vertical earthquake acceleration | .. | 0.1 g |

When checking the design for barrage for forces including earthquake factor, 33-1/3% extra over normal stress of t=18,000 lbs per sq. inch, and c=750 ibs per sq. inch have been allowed in the designs.

*Piers*: The piers have been designed for an earthquake acceleration of 0.15 g horizontal and 0.1 g vertical. As the structures are massive, the dynamic analysis has not been considered necessary and design was made assuming that the earthquake force is equivalent to a static force at the centre of gravity in either direction.

**5. Earth Dam and Embankments**

For the design of earth and rockfill dams, the most common method for evaluating the stability of slopes is by the Swedish Slip-Circle Method, which is based on the assumption that failure plane of an earth-slope is cylindrical. The method aims at the determination of the total shearing forces and the total shear resistance along an assumed cylindrical surface, thereby arriving at a factor of safety against sliding. The factor of safety is expressed as follows :

$$S_f = \frac{C(N-P)\,\mathrm{Tan}\,\phi}{T}$$

where

C=Cohesion;

N=Summation of forces normal to the sliding plane;

P=Pore pressure;

Tan $\phi$=Tangent of the angle of internal friction; and

T=Summation of forces tangential to the sliding plane.

Although it is generally recognized that an earth dam or embankment will vibrate when subject to ground motion during an earthquake, requiring thereby a dynamic analysis of the structure, nevertheless, currently accepted design procedures are based on the assumption that the structure is rigid. This is largely on account of the fact that a dynamic analysis, besides being a considerably more complicated procedure, involves the application of data which is seldom available and regarding which there is great uncertainty. Therefore, the seismic coefficient method as applied in masonry dam is adopted in the case of earthen dams also.

The earthquake effect is considered to be the worst when the reservoir is in the state of drawdown. The earthquake force is considered to act in a horizontal direction and the factor of safety is calculated by taking ratio of total shear resistance to develop shear force along the most unfavourable circular force of sliding. Besides the effects of (a) change in horizontal and vertical components of reservoir forces, (b) horizontal inertia due to horizontal earthquake acceleration, (c) change in unit weight of earth and water due to vertical acceleration, are also considered. The pore pressure is assumed to remain ineffective. The value of cohesion is doubled with no change in the angle of internal friction.

**6. Stability of Upstream Slopes**

The stability of the upstream slope of an earth or rockfill dam is tested with reservoir level at full operating elevation with horizontal earthquake acceleration acting in a direction which yields the most adverse conditions of stability. The maximum drawdown condition is not combined with earthquake forces. The stability of the upstream slope is also tested with reservoir level depleted to the minimum operating elevation under steady state of seepage conditions for the above mentioned earthquake accelerations. The hydrostatic reservoir load is then reduced to the extent of hydrodynamic pressure determined in the manner described earlier and modified on account of vertical earthquake acceleration.

An additional horizontal inertia force due to horizontal earthquake acceleration is then applied to the earth mass above the plane of the slope acting in a direction towards the reservoir. The unit weight of the soil mass above the plane of the slip for computing the inertia force is the saturated weight of the soil mass lying below the pheratic line and the moist or dry weight of the soil mass lying above that line. For determining the normal and tangential forces along the plane of the slip, the unit weight of the soil mass above the plane of the slip as adopted for the non-seismic analysis are increased or decreased to account for the vertical earthquake acceleration as in the case of masonry dams.

The values for cohesion and the angle of internal friction of the soil for the seismic analysis are considered to be the same as in the case of static analysis.

**7. Stability of Downstream Slope**

The stability of the downstream slope is tested wih reservoir level at normal operating elevation as in the case of non-seismic analysis. The soil mass above the assumed plane of slip is subjected to horizontal earthquake acceleration in an upstream direction and a vertical acceleration which yields most adverse conditions of stability. For determining the horizontal inertia force, applied in a downstream direction, the saturated unit weight of the soil mass lying below the pheratic line and moist or dry unit weights of the soil above that line are adopted.

A few examples of earth dams designed to resist the earthquake forces are given below:

1. *Umiam Barapani Hydel Project*

The Umiam Barapani Project consists of a concrete dam with some length of earthen dyke. The dyke is about 84 ft. high at the deepest section. It has an upstream slope varying from 2:1 to 4:1 and d/s slope varying from 2 : 1 to 3 : 1 with suitable berms. The dam is situated in seismically active zone and hence it is designed to withstand earthquake accelerations of 0.225 g horizontal and 0.1 g vertical. Both horizontal and vertical accelerations are assumed to act simultaneously.

The critical condition for upstream slope is assumed to be sudden drawdown from maximum W.S. El. 3210 to dead storage level 3150, combined with earthquake shocks.

While analysing the upstream slope, additional horizontal earthquake acceleration is applied to the mass of earth above the plane of slip acting in a direction towards the reservoir. For computing the component of inertia force parallel to tangential force, the saturated unit weight for the soil mass below the pheratic line and moist weight for that lying above the line is taken. For component parallel to normal force submerged unit weight is taken.

To account for the vertical earthquake acceleration, the normal force is decreased or increased depending upon the direction of earthquake.

The pore pressure is assumed to be unaffected during the earthquake. Under earthquake condition the value of cohesion is doubled but the angle of internal friction is assumed to be unaltered.

Factor of safety above unity is accepted as being adequate for ensuring stability of the upstream slope. The actual factor of safety obtained is 1.15.

*Downstream slope*

The critical condition for downstream slope is assumed to be maximum W. S. 3210 on upstream and no tailwater.

The inertia force in this case is applied in a downstream direction. For determining the component of inertia force parallel to tangential force the saturated unit weight for the soil mass below the pheratic line and moist weight above the line is adopted. For component parallel to normal force submerged unit weight is adopted.

In all the above cases, only dry unit weight is taken for rockfill portion of the dam. The normal forces are increased or decreased to account for vertical earthquake acceleration. The minimum safety factor obtained is 1.12.

2. *Kosi Project*

The dam is situated in an active earthquake zone and therefore it is designed to withstand the earthquake accelerations of 0.2 g horizontal and 0.1 g vertical. The stability of the upstream and downstream slopes of the dam is analysed by slip circle method in a manner similar to Umiam Barapani Project.

3. *Obra Project*

Obra Dam consists of earth and rockfill dam with a spillway on either flank and a dyke on its left flank. The dam lies in the seismic zone No. VIII corresponding to an earthquake of G/20 (according to Geological Survey of India map). For the design of this dam, however, an earthquake of G/10 intensity has been assumed. Stability effect due to enhanced hydrodynamic pressure during earthquake has also been accounted for.

The dam is about 91.5 ft. in height and has a top width of 30′. The upstream slope is 2:1 with berms 15′ wide. On the downstream also, the slope is 2:1 with berms of 15′. Near the toe of the downstream side the slope is increased to 2.5:1. Earthquake acceleration with sudden drawdown condition has also been analysed.

4. *Ramganga Dam*

Ramganga Dam across Ramganga river in Uttar Pradesh will be a 412 ft. high earth dam. The proposed dam will be a zoned structure with an impervious core and casing of semipervious to pervious materials. The area in which the dam will be constructed is highly seismic. It has been estimated that the maximum intensity may be of the order of No. VIII of modified Mercalli Scale which corresponds to horizontal accelerations from 50-330 cm/sec$^2$ (0.059 to 0.239). Considering the effect due to the nature of the foundation which consists of river bed material and soft rocks, coefficients of horizontal and vertical acceleration were taken as 0.2 g and 0.1 g respectively, in the design. The upstream slope was tested for the condition of sudden drawdown from MWL 1200 to El. 1063 and the downstream slope for the condition upstream reservoir elevation MWL 1200 and no tail-water.

The minimum factors of safety under various conditions were as below :

| | | |
|---|---|---|
| (i) | Without earthquake .. .. .. | 1.5 |
| (ii) | With earthquake and neglecting cohesion (Reservoir full) .. .. .. | 1.1 |
| (iii) | With earthquake neglecting cohesion (Drawdown) .. .. .. .. | 1.1 |
| (iv) | With earthquake with cohesion (Drawdown) .. .. .. .. .. | 1.1 |

*APPENDIX P*

## PROCEDURE FOR EARTHQUAKE RESISTANT DESIGNS OF DAMS—ABROAD

### 1. Introduction

The shock waves set up in the earth's outer crust as a result of sudden adjustment of the internal stresses are known an earthquakes. There are three predominant types of earthquake waves recognised and recorded. They are longitudinal (P) waves, transverse (S) waves and ground (R & L) waves. Of these 'P' waves, which have the highest velocity of travel are recorded first. 'S' waves are slower than 'P' waves and are recorded next. Ground waves 'R & L' are recorded last. They are slower than 'S' waves.

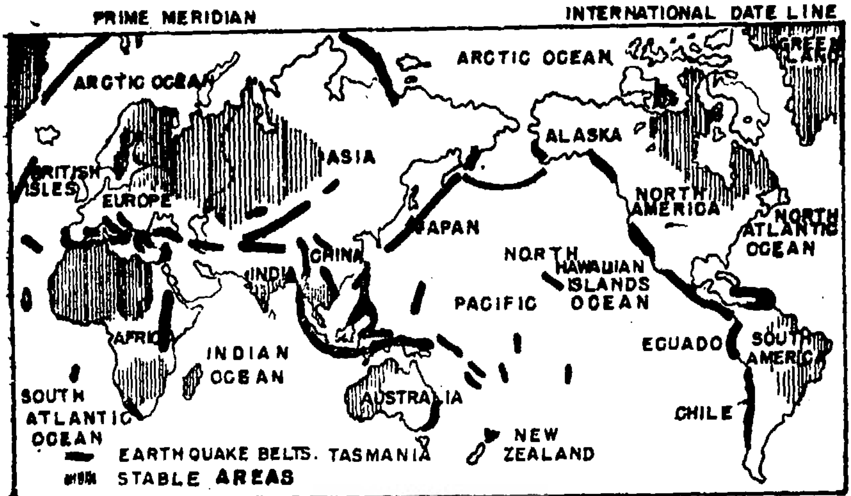
Earthquakes are experienced practically in all parts of the world. However, there are some regions of the world where the occurrence of earthquakes is a frequent feature. The principal earthquake belts considered by Gutenberg and Richter as given by J. M. Rapheal in his article "Design of Dams for Earthquake Resistance" presented at World Conference on Earthquake Engineering held at California in 1956 are :

1. The Circum-Pacific belt : This extends around the entire rim of the Pacific Ocean.
2. The Alpine belt : This extends from the Azores, across the Alpine structures of Mediterranean Europe, and across Asia to Burma, along the front of the Himalayas.
3. Minor active areas, including narrow belts in the Arctic, Atlantic and Indian Oceans, rift zones internal to the stable masses and active areas marginal to the continental stable masses.
4. Stable masses : The continental nuclei of old rocks—small shocks occur even here and seem to take place occasionally almost anywhere.

Earthquake belts in the world are shown in Fig. No. 1.

### 2. Damage to Dam Structures

It may be stated that, by and large, a number of dams built in the seismic zones of the various parts of the world have withstood the earthquake shocks well. But instances are not lacking where the structures have been damaged to an appreciable extent by earthquakes. For example, San Andreas Dam in California, U.S.A. which is an earth-fill dam 95′ high, moved bodily by 8 feet in 1906 earthquake. Similarly, upper crystal springs dam in California, an earth-fill dam 85′ high, developed several shear cracks along the fault in 1906 earthquake. One part of the dam moved by 8 ft. and further cracks were developed parallel to its axis. The experience in Japan indicates that very little damage was experienced in the case of




EARTHQUAKE BELTS AND STABLE AREAS OF THE WORLD
(APPENDIX . P )

FIG. 1

concrete dams, but there were settlements and longitudinal cracks in earth dams. An analysis of Ministry of Construction, Japan reveals that there were 43 instances of axial cracks, 17 slidings of upstream slopes, 6 of downstream slopes and 8 on both sides.

Therefore, it is necessary that the dams built in seismic zones must be designed to withstand forces set up during apprehended earthquake. General design criteria are described below.

### 3. General Design Criteria

Many attempts have been made to analyse mathematically and laboratory tests have been conducted to study the precise nature of the earthquake forces, but even to this date, these are not fully explored. Earthquake shocks subject dams to dynamic forces. But in current practice of the design, the dynamic problem is converted to one of the statics. Briefly the general practice in the statical analysis is to make an allowance for the effect of increased water and silt pressure and an additional seismic allowance for the horizontal and vertical stresses the earthquake accelerations cause within the dam. The allowance to be made depends upon the accelerations imparted by the earthquake to the earth's crust. The acceleration considered is expressed as a ratio of the earthquake acceleration to the gravity acceleration and is known as seismic coefficient or earthquake factor. This method is, however, inadequate and falls short of scientific exactness. Hence, there is a growing practice to conduct model tests for dynamic analysis simulating the earthquake effects to evaluate the stresses developed within the structure. In short, the procedure for a seismic design of a dam comprises :

(i) to fix the seismic coefficient; and then,

(ii) to check the stability of the structure under the worst combination of normal forces and forces set up by the earthquake; and

(iii) also if possible to conduct model studies for important dams simulating the earthquake effects to evaluate the stresses and examine the stability of the dam.

The two main components of earthquake acceleration are horizontal and vertical. In the case of a gravity dam the direction of motion parallel to the axis is generally not considered, as this is not likely to give rise to most unstable condition of the section. For a dam with reservoir full, the most unfavourable direction is upstream normal to the axis. Under this condition, in addition to the inertia force acting downstream, there is the hydro-dynamic pressure tending to overturn the dam. For reservoir empty, a downstream acceleration, with corresponding force acting upstream is more unfavourable.

The horizontal acceleration, which is responsible for most damage, is accounted for in the seismic design of dams in all the countries. However, as regards the vertical acceleration there is a difference of approach. As indicated by J. K. Hunter and H. C. Keefe in the Paper "Special problems relating to the construction of dams in active volcanic country", it is generally considered unnecessary to take account of a vertical acceleration. According to the same authors, in the first place the vertical acceleration set up by the average earthquake will, unless the dam is close to the epicentre, be small compared with the horizontal component. Moreover in masonry dam the primary effect of such vertical acceleration is merely to vary the apparent specific gravities of the dam and the water without altering their ratio.

In Japan, the prevailing practice as seen from the examples of the dams built in that country, is to neglect the effects of vertical acceleration. In America, in the design of dams like Friant, American Falls, East Park etc; the combined effect of horizontal and vertical earthquake is considered. In these dams, both horizontal and vertical accelerations of same value have been taken into account. In India too, both vertical and horizontal accelerations are considered simultaneously. The value of vertical acceleration is, however, assumed to be generally half that of horizontal acceleration.

It is to be observed that earthquakes are of short duration and are of infrequent occurrence. Further, it may not normally be possible to save a structure in the eventuality of an earthquake of such magnitude as to cause a complete disruption. For these reasons a seismic design is greatly influenced by economy. In fact, concessions up to 10 to 20 per cent in the permissible stresses and relaxation in the factors of safety are allowed where the design is checked for a condition of maximum earthquake, combined with the most adverse combination of other conditions. In Japan, a number of arch and gravity dams have been designed where the allowable stresses have been increased considerably. Even so, a doubt is expressed in Japan about the wisdom of relaxing the safety factors and permissible stresses when the effect of earthquake forces has not been fully revealed as yet. Table 1 gives a list of dams in Japan, in which the allowable stress was increased for earthquake conditions.

TABLE 1

*Dams in Japan in which the Allowable Stress was Increased*

| Name of dam | Height in metres | Normal allowable stress (kg/cm²) | Rate of increase | Type of dam | Remarks |
|---|---|---|---|---|---|
| Gozenzawa .. | 190·00 | 84 | 15 | Arch Dam | |
| Sasanamigawa .. | 67·40 | 55 | 15 | Arch Dam | |
| Sakuma .. .. | 155·50 | 50 | 15 | Gravity Dam | |
| Nukabira .. .. | 78·00 | 50 | 15 | Gravity Dam | |
| Tagokura .. .. | 145·00 | 50 | 15 | Gravity Dam | |
| Okutadami .. .. | 150·00 | 45 | 15 | Gravity Dam | |
| Kazaya .. .. | 100·00 | 50 | 15 | Gravity Dam | |

In the seismic design of dams, combination of transient loads each of which has a remote possibility of simultaneous occurrence is not taken into account. For example, earthquake is not considered to occur simultaneously with the maximum flood condition.

Experiments indicate that resistivity of soils increases during dynamic loading. Casagrande and Shnannon found that the dynamic shearing resistance of clays increased 40 to 160%, sands increased 20% and soft rock increased 80% over the static resistance. Hence in the design of earth dams for earthquake conditions the shearing resistance is generally increased by adopting as much as twice the value of cohesion.

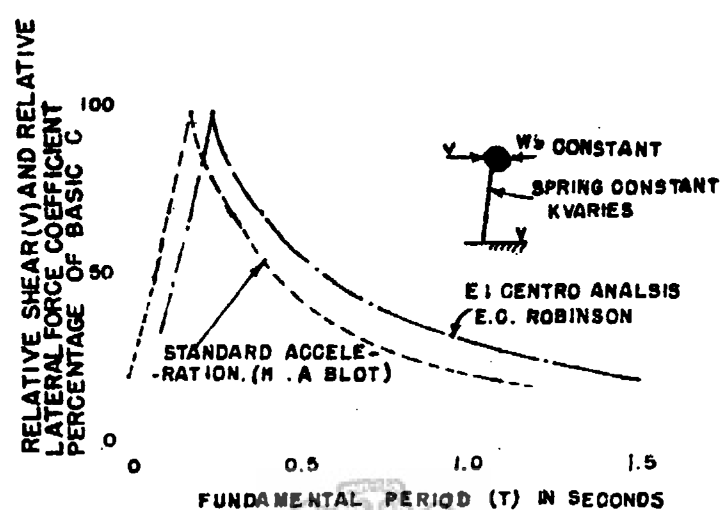
The seismic coefficient method described above makes the following assumptions:—

(i) a structure is of infinite rigidity and fails to consider the elastic characteristics of the structure,

(ii) the period of vibration is normally taken as 1 second, and

(iii) the ground motion due to earthquake is simple harmonic in nature.

Experience has shown that sinusoidal ground motion does not occur in the destructive zone of earthquake; nor the steady-state response assumed in the seismic coefficient method really exists.




RESPONSE OF SIMPLIFIED STRUCTURES TO EARTH QUAKES

FIG. 2

(APPENDIX . P)

In the absence of sustained harmonic vibrations, more elaborate dynamic theories are considered necessary. The earthquake spectrum is a dynamic approach to the determination of earthquake forces on structures, and may be defined as a graph exhibiting the effects of an individual earthquake on structures with different free periods of vibration. The period of vibration is plotted along the x-axis and the effect of earthquake expressed in terms of base shear, maximum acceleration Sa, or maximum velocity Sv along the y-axis (Fig. 2).

The undamped spectra of earthquake is only a qualitative indication of structural response which is never realised in actual structure due to effects of damping, tilting etc. which tend to reduce the acceleration experienced considerably. The effect of various degrees of damping has been investigated and the results are presented in Fig. 3. Knowing the period of the structure and the damping expressed as a percentage of the critical, the acceleration Sa, to be used is found from the spectrum (Fig. 3).

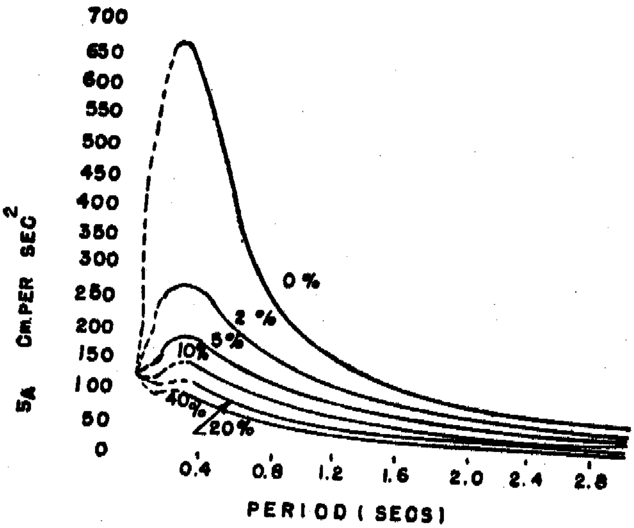
Experiments conducted in Japan have shown that the acceleration in a dam is not uniform and that at the crown it can be as much as 2 to 3 times that at the base. However, this effect is not yet being taken into cognizance in the design of dams.

**4. Seismic Coefficient**

The seismic coefficient is governed by the following factors :—

(i) type and life span of the dam;

(ii) nature of foundations;




AVERAGE ACCELERATION SPECTRUM CURVES

FIG. 3
(APPENDIX. P)

(iii) past history of magnitude of earthquakes and frequency of their occurrences; and

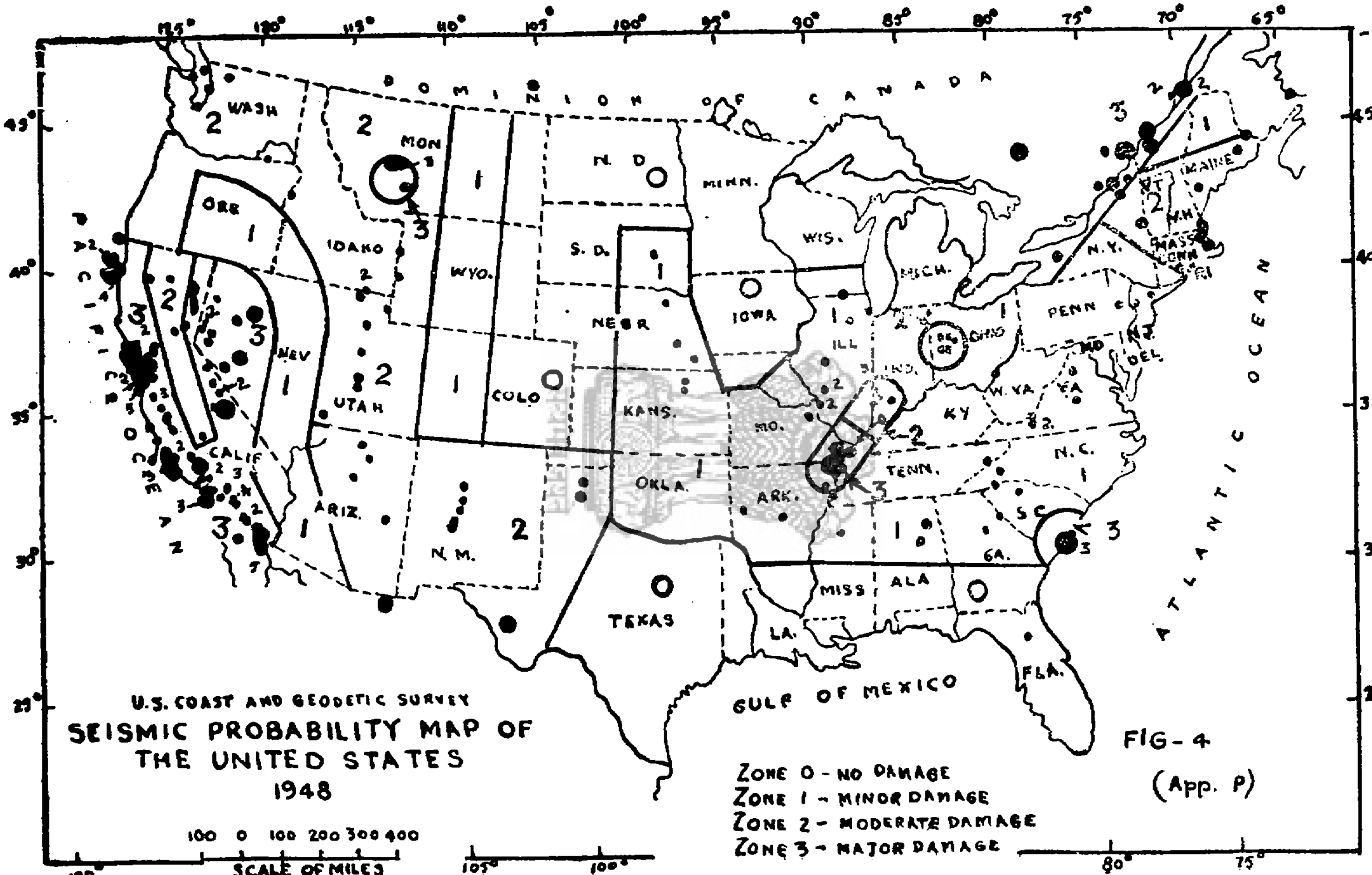
(iv) location of active faults and their distances from the structures.

The past history of earthquakes has been extensively studied in America and Japan and the seismic probability maps have been drawn up and earthquake factors to be adopted for design of structures in these zones laid down. But these are only guiding values, the actual value being determined for every important high dam by study of its features, geology and the seismic data of its location. The seismic probability map of America is shown at Fig. 4. In Japan, the seismic coefficient is determined within the range illustrated in Fig. 5 and Table 2, depending on the subsoil conditions and importance of the structure. Figures 6 & 7 show the dams affected and unaffected in Nankai and Kanto earthquakes respectively.

TABLE 2

*Design seismic coefficients in Japan*

| Type of dam | District | |
|---|---|---|
| | Tohoku region (Fukushima, Akita, Miyagi Prefactures); Kanto region; Chubu region; Kinki region; Southern Shikoku region | Hokkaido region; Hokuriku region; Tohoku region (Iwate, Yamagata, Amori Prefactures); Northern Shikoku region; Kyushu region |
| Concrete dams and rockfill dams .. | 0·12—0·20 | 0·10—0·15 |
| Earth dams .. .. .. .. .. | 0·15—0·25 | 0·12—0·20 |


U.S. COAST AND GEODETIC SURVEY
SEISMIC PROBABILITY MAP OF
THE UNITED STATES
1948
ZONE 0 - NO DAMAGE
ZONE 1 - MINOR DAMAGE
ZONE 2 - MODERATE DAMAGE
ZONE 3 - MAJOR DAMAGE
100 0 100 200 300 400
SCALE OF MILES
DOMINION OF CANADA
ATLANTIC OCEAN
PACIFIC OCEAN
GULF OF MEXICO
WASH
ORE
CALIF
NEV
IDAHO
MON
WYO.
UTAH
ARIZ.
COLO
N.M.
N. D
S. D.
NEBR
KANS.
OKLA.
TEXAS
MINN.
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VA
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PENN
N.Y.
MD
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MAINE
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FIG-4
(App. P)

In America and India the practice is to adopt the same value of horizontal acceleration, irrespective of the fact whether the reservoir is considered to be empty or full. In Japan, the recognised practice is to reduce the acceleration factor by half for the condition of reservoir empty. It is reasoned out that when the reservoir is empty the likely damage is considerably less than when it is full. Seismic coefficients used in some of the existing dams of Japan may be seen at Table 3.

TABLE 3

*Seismic coefficients used in some of the dams in Japan*

| Name of dam | Type of dam | Height in metres | Equivalent horizontal seismic | |
|---|---|---|---|---|
| | | | Full reservoir | Empty |
| Yakuwa | Gravity dam | 97·00 | 0·12 | 0·06 |
| Sudagai | „ | 73·00 | 0·12 | 0·06 |
| Asahi | „ | 83·00 | 0·12 | 0·06 |
| Akigami | „ | 71·50 | 0·12 | 0·06 |
| Arimine | „ | 140·00 | 0·12 | 0·06 |
| Maruyama | „ | 98·20 | 0·15 | 0·075 |
| Miura | „ | 84·10 | 0·15 | 0·075 |
| Komaki | „ | 75·00 | 0·15 | 0·075 |
| Odomari | „ | 74·00 | 0·12 | — |
| Yuharadaiichi | „ | 73·50 | 0·12 | 0·06 |
| Ohashi | „ | 73·50 | 0·12 | 0·06 |
| Tsukabaru | „ | 87·00 | 0·12 | 0·06 |
| Sakuma | „ | 155·50 | 0·12 | 0·06 |
| Nukabira | „ | 78·00 | 0·12 | 0·06 |
| Kuromatagawa | „ | 91·00 | 0·12 | 0·06 |
| Akiba | „ | 84·00 | 0·12 | 0·06 |
| Tagokura | „ | 145·00 | 0·12 | 0·06 |
| Nagayama | „ | 38·00 | 0·12 | 0·06 |
| Okugadami | „ | 157·00 | 0·12 | 0·06 |
| Kazaya | „ | 100·00 | 0·12 | 0·12 |
| Ikawa | Hollow gravity | 103·60 | 0·12 | 0·06 |
| Omorigawa | „ | 73·50 | 0·12 | 0·06 |
| Morotsuka | „ | 61·00 | 0·12 | 0·06 |
| Gozenzawa | Arch Dam | 190·00 | 0·12 | 0·12 |
| Tonoyama | „ | 64·50 | 0·12 | — |
| Sasanamigawa | „ | 67·40 | 0·12 | — |
| Kamishwiba | „ | 110·00 | 0·12 | 0·06 |
| Muromaki | „ | 82·00 | 0·12 | 0·06 |
| Ayakita | „ | 73·30 | 0·12 | 0·06 |



So far in this discussion, no attempt has been made to differentiate between the various types of dams. However, the structural action varies greatly according to the type of the dam and the material it is composed of. In the following paragraphs seismic designs for different types of dams, are considered separately.

**5. Seismic Design for Gravity Dams**

A gravity dam built in seismic zones is likely to be subjected to the following forces in addition to static forces which are generally considered for non-earthquake areas :

(a) Vibration of dam itself in the horizontal or vertical direction.

(b) Vibrating column of water immediately on the upstream of the dam and tailwater.

(c) Movement of silt and ice on upstream of the dam.

The horizontal inertia force of the body of the dam due to horizontal earthquake acceleration is a product of the weight of the dam and the earthquake acceleration factor. This inertia force acts in a direction opposite to the direction of the earthquake acceleration.

The primary effect of a vertical earthquake acceleration is merely to vary the apparent specific gravity of the dam and the water without altering their ratio. However, the stress under no earthquake condition would get multiplied by $(1 \pm \alpha)$ depending upon whether the earthquake vibration is downward or upward.

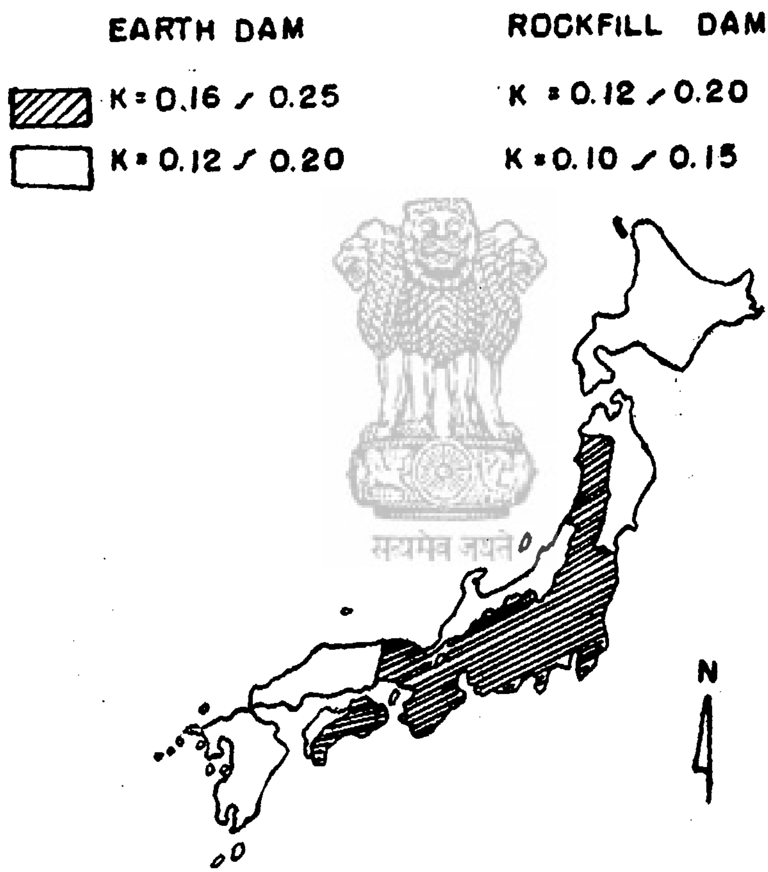
Due to the horizontal acceleration of the foundation and dam, there is an instantaneous hydro-dynamic pressure exerted against the dam. It is apparent that the direction of the hydro-dynamic force is opposite to the direction of earthquake acceleration. Prof. Westergaard and Zanger have studied the problem and have given easily applicable formulae for determination of the hydro-dynamic force.

It is generally believed that increase in pressure due to ice and silt during an earthquake is not of material importance on dams. This assumption is reasonable as the formation of ice is restricted within a few feet of the water surface and the depth of silt touching the upstream face of the dam is generally small.

Maximum destruction occurs when the effect of resonance is produced *i.e.* when the natural frequency of the structure equals the prevailing frequency of the earthquake elastic waves. Westergaard has determined the free period of vibration for a concrete gravity dam of triangular section considering the reservoir empty and expressed it as $t_s = \frac{h^2}{2000\, a}$ where '$t_s$' is the time period in seconds, 'h' is the height of the dam in ft. and 'a' the length of the base in feet. It is seen from this equation that possibility of resonance in dams is not of importance for heights less than 1000 ft. Further, the various forms of damping, some of which are difficult to evaluate, act to prevent resonance. Hence the resonance effect is not normally taken into account.

Uplift force on a dam is a function of hydro-static pressure of reservoir and tailwater. During an earthquake, the water pressure is not considered effective in producing a corresponding increase or reduction in the uplift force. The duration of the earthquake is too short to permit the building up of pore pressure in the body of the dam and foundations.

As already mentioned earlier, for reasons of economy and also because of the rare occurrence of earthquakes, certain concessions are made in the design criteria. The permissible stresses are increased by 10 to 20%. It is obvious that the worst condition for the failure of dam is when the earthquake shocks are felt during maximum flood condition. Because of the remote possibility of such simultaneous occurrence, earthquake is not considered along with maximum flood condition for the purpose of seismic design.






MAP OF DESIGN SEISMIC-COEFFICIENT K - (DAMS)

FIG. 5 (APPENDIX P)


o - - - DAM UNAFFECTED
• - - - DAM DAMAGED
V–VI - - JMA SEISMIC INTENSITY SCALE

LOCATION MAP OF DAMS IN EXISTENCE AT THE NANKAI EARTH QUAKE
AND DISTRIBNOTION OF SEISMIC INTENSITY
FIG - 6 (APP. P)

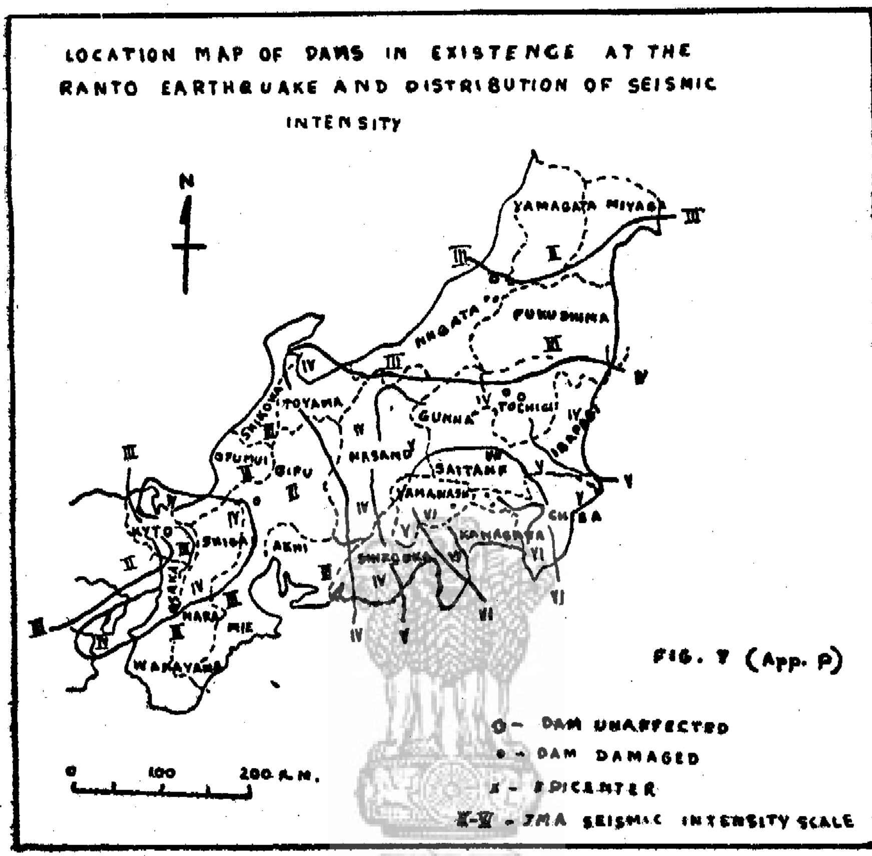



FIG. 7 (App. P)

In Japan and elsewhere a number of dams have been built in active seismic zones, the design criteria of which are fundamentally based on the above principles. In the following paragraphs a few such examples from Japan and America are quoted, describing briefly different load combinations, seismic coefficients adopted etc., followed by a note summarising the general procedure for the seismic design.

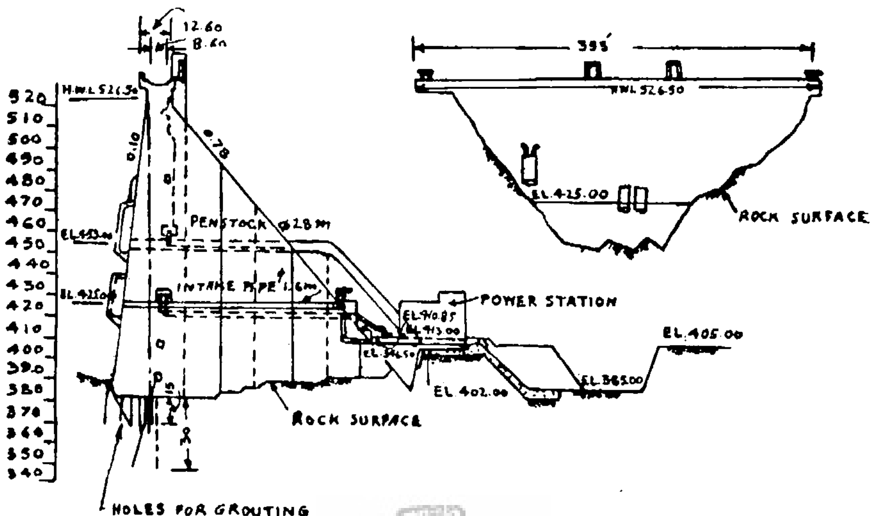
(1) *Ogochi Dam—Japan*

The Ogochi Dam is a non-overflow concrete straight gravity structure constructed in the upper reaches of the Tama River in the Kanto region. The dam is 149 m high, 353 m long at the crest and the volume of concrete is 1,675,680 $m^3$ (Fig. 8). This dam was built by the block system and keys were provided in both the longitudinal and transverse joints which were bonded by grouting after cooling of the mass concrete. Stress calculations made in the design of the dam were conducted under the following combinations of loads:

(a) Static water pressure at full water level, silt pressure, earthquake force in the downstream direction, dynamic water pressure, dead load and uplift pressure.

(b) Static water pressure at full water level, silt pressure, dead load and uplift pressure.

(c) Dead load, and earthquake force in the upstream direction.




OGOCHI DAM (CONC. GRAVITY TYPE)

FIG. 8 (APP. P)

In the trial load analysis, variations of stress conditions in ungrouted and grouted transverse joints and of other conditions were analysed.

The equivalent horizontal seismic coefficient taken for full reservoir condition was 0.12 and for empty reservoir 0.06. The allowable compressive stress of concrete under earthquake condition was taken as 46 kg/cm$^2$. To study the results of the design calqulations, model tests were made by the photo-elastic method. However, in this analysis, the earthquake force was considered as an external static force. Studies are being still continued on the behaviour of the dam against the vibration caused by natural earthquakes.

(2) *Ikawa Dam—Japan*

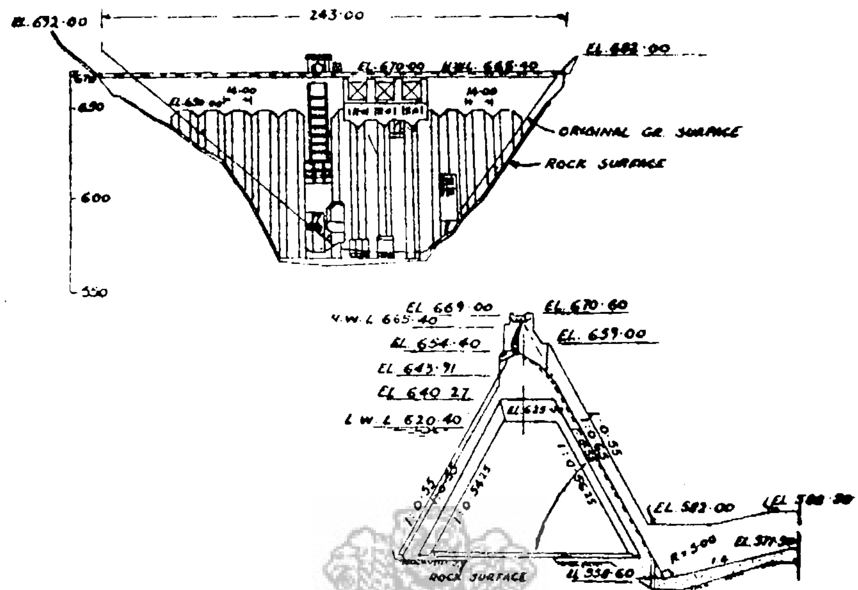
The Ikawa Dam is a hollow gravity type concrete dam, 103.6 m high and 243 m long at the crest, constructed in the upper reaches of the Oi River in the Chubu region (Fig. 9). The upstream and downstream slopes are 1 : 0.55 and the section is I shaped.

Studies on the seismic design of this dam were carried out for the following load combinations :—

(a) Static water pressure at full water level, silt pressure, earthquake force in the downstream direction, dynamic water pressure and the dead load.

(b) Earthquake force in the upstream direction and the dead load.

(c) Earthquake force in the direction of the dam axis and the dead load.



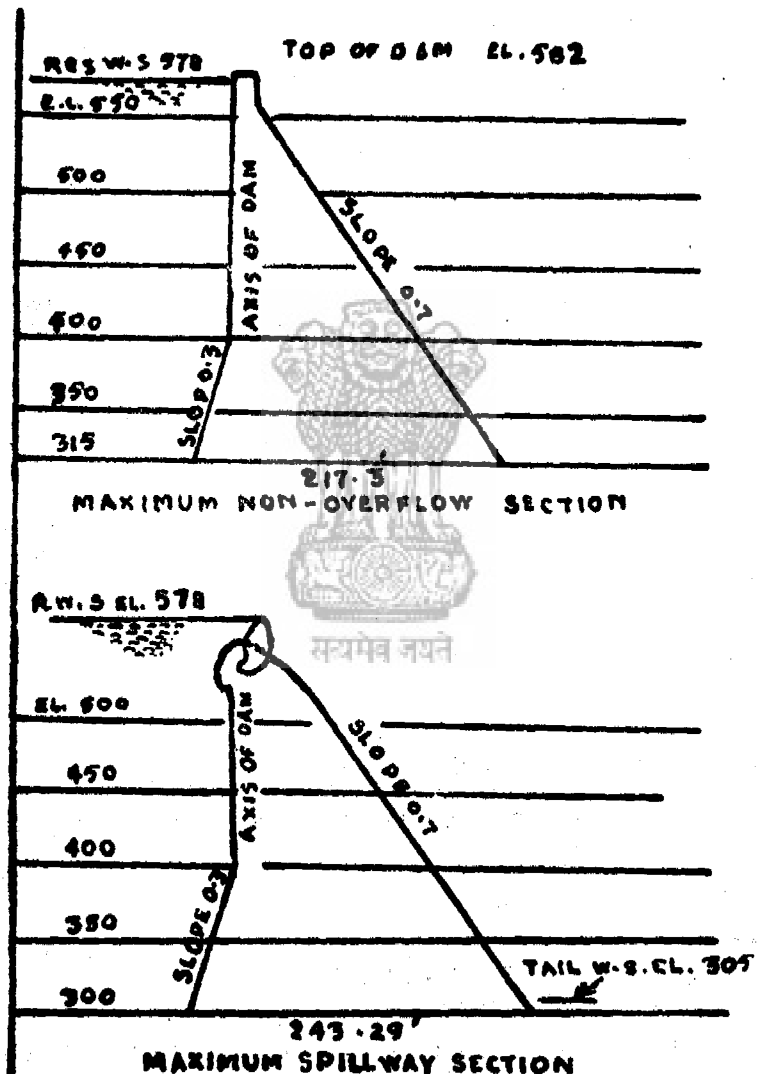
IKAWA DAM SECTIONS & ELEVATION
FIG. 9
(APP. P)

The equivalent horizontal seismic coefficient taken for full water condition was 0.21 and for the empty reservoir 0.07. For stress calculations, in the case of (a) and (b) load conditions, the load distribution was assumed trapezoidal and in the case of (c), the buttress wall was assumed to be a fixed triangular slab. The stress distribution was studied by three dimensional photo-elastic experiments to check and correct the computed values.

From the standpoint of resistivity to earthquake, case (c) was considered to be the most important one because of the existence of tensile stress, though very little, and the vibration experiments on rubber models were supplemented to determine the period and mode of natural vibration. The results of these experiments generally agreed with the calculated values and the primary vibration period of the highest section in the actual dam was estimated to be in the range of 0.11 to 0.12 second.

After the dam was completed, the vibration test with vibrators was conducted and also the vibrations of the dam and its foundation caused by natural earthquakes have been recorded.

The vibration experiments were primarily conducted to study the vibration characteristics in the direction of the dam axis. The vibration period is generally close to the value obtained in the model experiments and the value of damping constant is extremely small, being about 2%. In view of the fact that the fundamental vibration period of the dam is short in comparison with the predominant period of the earthquake motion and consequently, the degree of resonance being rather small, it was concluded that the dam is stable against earthquake. Vibration records of natural earthquakes are generally in agreement with the estimated values.






FRIANT DAM

FIG. 10
(App. F)

(3) *Friant Dam—U.S.A.*

Friant Dam located in the Central Valley of California, was constructed during the period 1939 to 1942. The non-overflow section is 267 feet high at the deepest foundation. The maximum spillway section may be seen at Fig. 10.

The design of the Friant Dam was influenced by earthquake considerations which are as follows:

(1) The horizontal component of earthquake acceleration was assumed as 0.1 g, a period of vibration of 1 sec. and the direction of acceleration at right angles to axis of dam.

(2) The vertical acceleration assumed as 0.1 g and a period of vibration 1 sec.

(3) For combined effects, the horizontal and vertical accelerations were assumed to occur simultaneously.

Loads for earthquake effects with reservoir empty include inertia forces caused by acceleration of the mass of the dead loads. Loads for earthquake effects with reservoir full include, in addition to the above, the inertia force of the mass of water and the hydro-dynamic force caused by the movement of the dam against the water of the reservoir.

Uplift forces are assumed to be unaffected by earthquake shocks.

The effects of earthquake were studied for each of the following directions :—

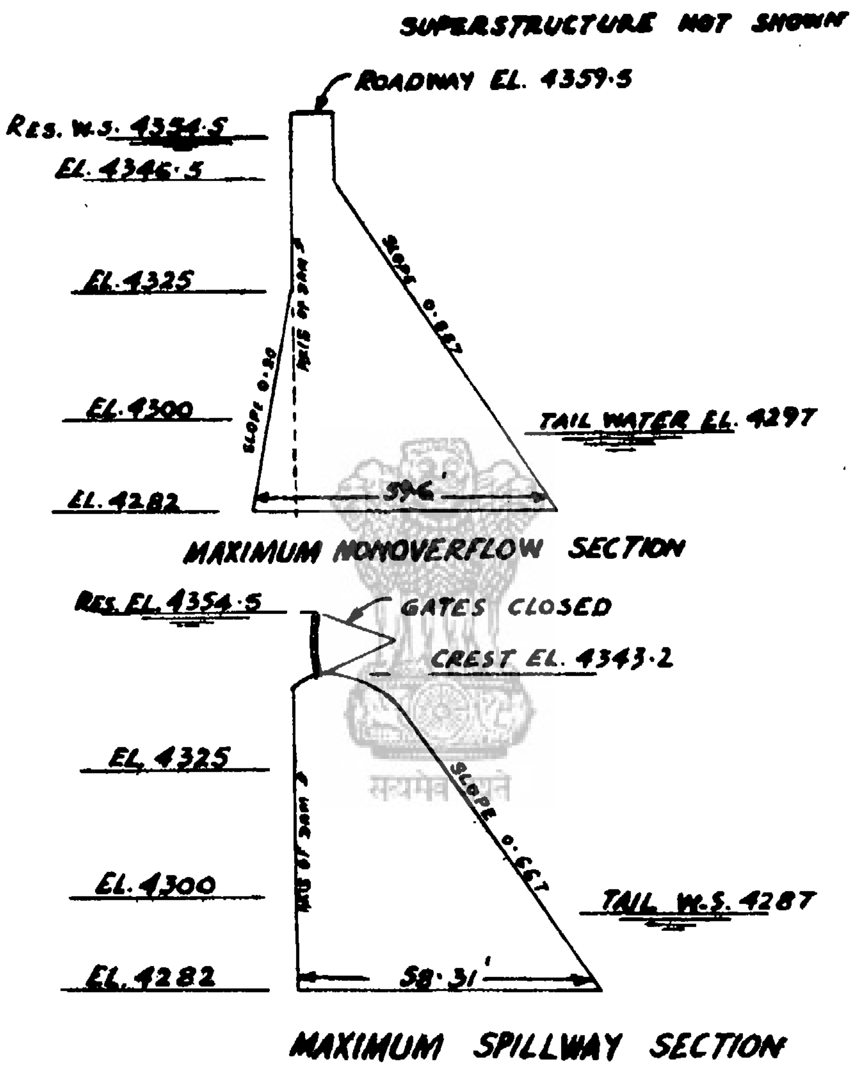
(i) Horizontal upstream.
(ii) Horizontal downstream.
(iii) Vertical upward.
(iv) Vertical downward.
(v) Horizontal upstream plus vertical upward.
(vi) Horizontal upstream plus vertical downward.
(vii) Horizontal downstream plus vertical upward.
(viii) Horizontal downstream plus vertical downward.

Analysis under different conditions mentioned above has shown that the maximum compressive stress, maximum horizontal shear stress, and minimum shear friction factor all occur for normal full reservoir operation during earthquake acceleration "horizontal upstream" and "vertical upward".

Earthquake shocks are not combined with maximum flood condition.

(4) *American Falls Dam—U.S.A.*

American Falls Dam is a straight gravity dam constructed in the year 1927 on the Snake River. It has a maximum height of 77.5 ft. and a crest length of 5227 ft. (Fig. 11).




AMERICAN FALLS DAM

FIG. 11
(APP. P)

Earthquake accelerations both horizontal and vertical, in combination with each other have been considered.

The load combinations for which the stability of the dam is analysed taking into account the earthquake accelerations are as follows :—

(i) Concrete weight and earthquake (vertical upward and horizontal downstream).

(ii) Water pressure, weight and earthquake (vertical upward and horizontal upstream).

(iii) Water pressure, weight, uplift and earthquake (vertical upward and horizontal upstream).

The maximum stress occurs for the condition (iii), when the reservoir is at normal full reservoir operation level. Earthquake effect is not combined with maximum flood condition.

(5) *Atlus Dam—U.S.A.*

It is a straight gravity dam constructed in the year 1945 on Red River, Oklahoma. The maximum section is 102 feet high. The length of the dam at crest is 1,112 ft.

Earthquake acceleration, both horizontal and vertical, supposed to act simultaneously are considered while checking the stability of the dam.

The different load combinations when earthquake shocks are considered are as follows :—

(i) Weight and earthquake (vertical upward and horizontal downstream).

(ii) Water pressure, weight and earthquake (vertical upward and horizontal upstream).

(iii) Water pressure, weight, uplift and earthquake (vertical upward and horizontal upstream).

The maximum stresses occur for the load combination (iii), when the reservoir is at normal full reservoir operation level. Earthquake effects are not combined with maximum flood condition.

(6) *East Park Dam—U.S.A.*

It is a curved gravity dam of maximum height 130 feet. The dam is situated in Little Stony Creek, California. The construction of the dam was completed by 1910. It has a crest length of 250 feet. The thickness at the top is 10.6′ and the base width is 90.6′.

In the stability analysis of the dam both the horizontal and vertical earthquake accelerations were taken into account.

The different load combinations for which the section was tested is as follows :—

(i) Weight and earthquake (vertical upward and horizontal downstream).

(ii) Weight, water pressure and earthquake (vertical upward and horizontal upstream).

(iii) Water pressure, weight, uplift and earthquake (vertical upward and horizontal upstream).

The worst conditions occur under load combination (iii). The earthquake effects are considered only with normal reservoir operation level and not with maximum flood condition.

**6. Summary of Experience in U.S.A. and Japan**

From a study of the dams mentioned in the foregoing paragraphs, it would be seen that on the following points there is identity of approach in America as well as in Japan:

1. Earthquake and maximum flood condition are not combined.
2. Resonance effect is not considered.
3. Uplift is assumed to be unaffected by earthquake vibration.
4. Reservoir is at F.R.L. when the earthquake force acts in the downstream direction. The resulting appropriate hydrodynamic pressures are considered.
5. The reservoir is assumed to be at low water level or empty when the direction of earthquake force is upstream.

It is further seen that on the points mentioned below, the approach is not the same in the two countries:

1. In America, vertical and horizontal earthquake accelerations, both of the same value and supposed to act simultaneously, have been considered. However, the vertical accelerations are not considered at all in the design of dams in Japan.
2. For the conditions of reservoir, empty and full, in America, the same value of horizontal seismic coefficient is used; whereas in Japan, it appears to be the practice to adopt half the value of horizontal acceleration for the condition—reservoir empty.
3. The acceleration parallel to the axis of the dam is not considered in America. In Japan, in the case of Ikawa Hollow Gravity Dam, horizontal acceleration in the direction of dam axis is also considered.

**7. Seismic Design of Earth and Rockfill Dams**

Due to the elasticity of the materials of which these dams are built, the effect of earthquake acceleration on earth and rockfill dams is much less serious than on solid gravity dams. Experience indicates that in adequately designed earthfill dam, the inherent factor of safety against slips, both upstream and downstream, is ample to provide against earthquake shocks. Till recently, it was the general practice to make liberal provisions in the design of earth dams for earthquake conditions. Precautions which are usually taken include:

(a) A liberal free board.

(b) A deep trench into the foundation to intercept water flow through foundation cracks.

(c) Making impervious section wider.

(d) Weighting down of impervious section with coarse pervious material.

(e) Provision of flatter slopes.

(f) Selection of such construction material as will have self-sealing properties.

However, with the advancement in the knowledge of soil mechanics and also with the development of slip-circle method of stability analysis for earth dams, a mathematical approach to the seismic design of earth dams is being attempted in recent times. The additional forces that are set up in any earth dam due to earthquake vibrations may be briefly stated as follows:—

(a) Increased pressure due to the reservoir water.

(b) Inertia force of the body of the dam.

(c) Increase in pore water pressure.

(d) Increased stress due to resonance.

The stability of the slopes of the earth dam section is evaluated by the Swedish Slip Circle method. Here the failure is assumed to occur along the arc of a circle, and forces that cause a mass of earth above the failure to move are evaluated against the forces along the arc that resist the movement. The safety factor is expressed as:

$$Sf = \frac{C + \text{Tan}\,\phi\,(\Sigma N - P)}{\Sigma T}$$

where,

Sf = Safety factor,

C = Cohesion,

Tan $\phi$ = Tangent of the angle of internal friction,

$\Sigma N$ = Summation of normal forces,

P = Pore pressure, and

$\Sigma T$ = Summation of tangential forces.

An earth dam will vibrate when subjected to ground motion during an earthquake. It thus requires a dynamic analysis of the structure. Nevertheless, the current practice is to assume that the structure is rigid and proceed along the seismic coefficient method as applied to masonry dams.

The forces (a) & (b) mentioned above are calculated in a manner identical to the masonry dams. The increase in the pore pressure is estimated by the following equation:

$$P+dp=\frac{Pa\ (\Delta+d\Delta)}{Va+hVw-(\Delta+d\Delta)}$$

where,

P=pore pressure,

Pa=atmospheric pressure,

Va=initial free air content,

$\Delta$=consolidation due to increased stress,

Vw=specific volume of water, and

h=0.0197.

These forces are readily resolved by the slip-circle method of stability analysis. However, the problem of resonance by its very nature cannot be applied to the slip-circle method of analysis.

The critical condition for the upstream slope of the dam occurs when the earthquake condition is combined with sudden drawdown. Under such condition of analysis, the dynamic water pressure, which does not exist is not taken into account. For the downstream slope, the worst condition occurs when earthquake is combined with reservoir at maximum water level. The inertia force of the dam is assumed to act towards the reservoir in the former case and away from the reservoir in the latter.

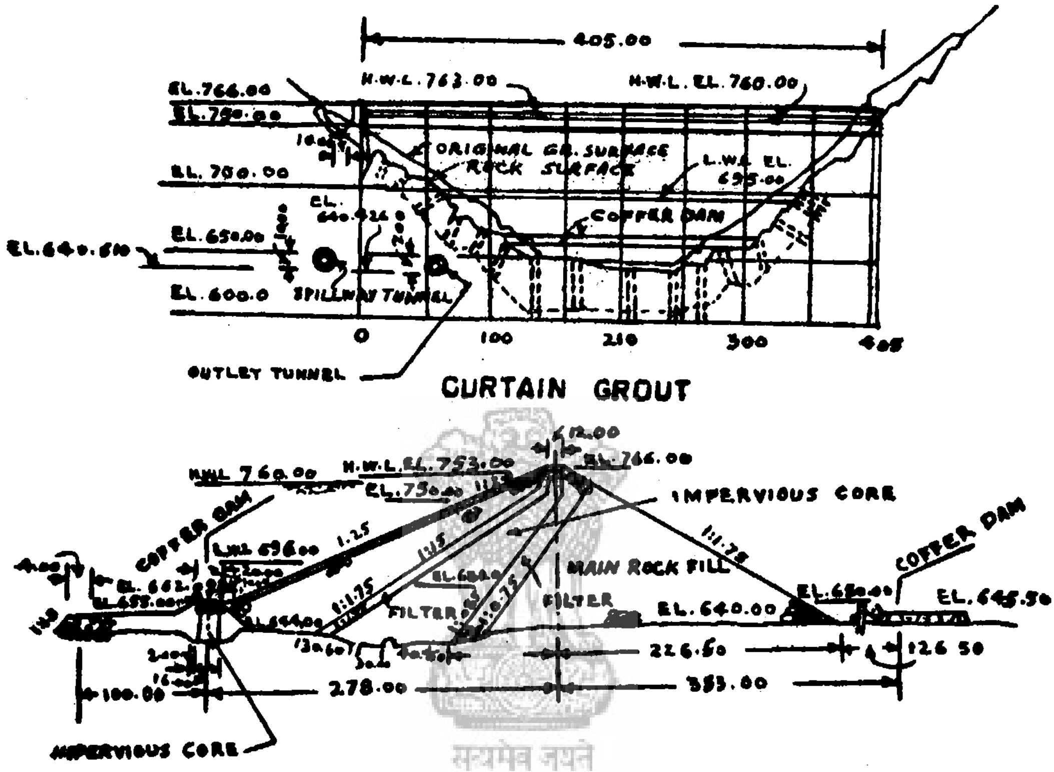
Experiments by Casagrande indicate that cohesive material may be as much as twice as resistant to transient dynamic stresses as to static stresses. Since the shear angle is nearly constant the cohesion must increase. Thus for earthquake condition, double the value of cohesion is applied to the equation for safety factor.

The possibility of resonance is the greatest risk to the stability of an earth structure. To a limited degree, the probability of structural resonance with prevalent ground frequencies can be mitigated by changing the height of the dam, by increasing the relative damping coefficient etc. The procedure of seismic design of an earth dam, both the conventional method of stability analysis and the dynamic method considering the resonance effects, is exhaustively treated in the technical memorandum No. 641 of U.S.B.R., some extracts of which are given in Annexure I.

In the following paragraphs, examples of a few earth and rockfill dams in Japan and America are described briefly. One example from Burma is also included.

1. *Miboro Dam—Japan*

.The Miboro Dam which is located in the upper reaches of the Sho River in the Hokuriku region is of a rockfill type with an inclined impervious clay core. The dam is 131 m high, 405 m long at the crest and the total earthfill is 7,950,000 $m^3$ (Fig. 12). Design calculations were made for the following load conditions.






**MIBORD DAM- SECTION & PROFILE**
**FIG. 12. (APP. P)**

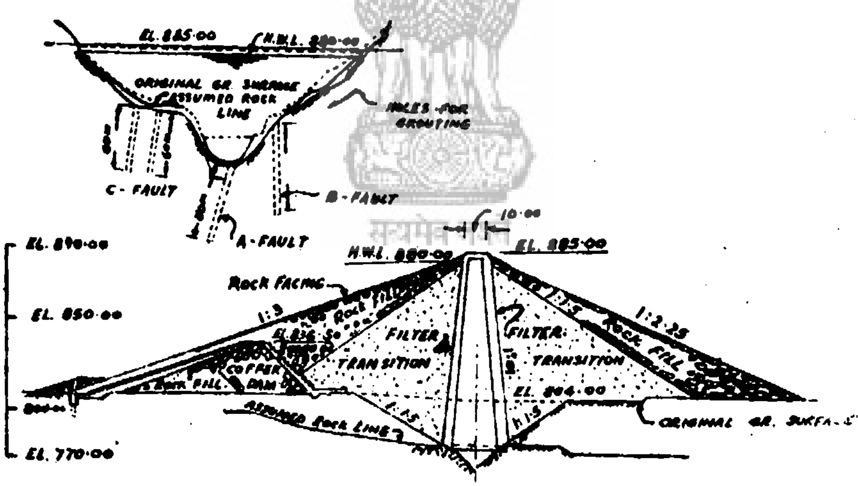
Stability calculations against sliding were made of the dam body (downstream filter zone and rock zone) under the full reservoir condition. Stability against bearing strength under full reservoir condition was examined at the base vertically below the downstream end of the crest of the standard section of the dam. On the stability of the upstream slope, calculations were made by the slip-circle method under the following conditions: at completion of the dam, full reservoir level and low reservoir level. Stability calculations of the downstream slope was also conducted by the slip-circle method. For the foundation, the stability against horizontal shearing force was calculated under the empty and full reservoir conditions. In all the stability studies described above, calculations were made taking into account the earthquake influence as well as not taking into account such influence.

Of the load conditions that were taken into account in the design calculations, the horizontal seismic coefficient was Kh=0.12 and in case of a temporary condition only one half of this value was considered.

As a result of these calculations, the minimum safety factor obtained for sliding was 1.1 and for stability of the slopes was 1.2.

(2) *Makio Dam—Japan*

The Makio Dam under construction on the tributary of the Kiso River in Nangano Prefacture will store water for the Aichi Irrigation System. This dam is a rockfill structure with a central clay core. The dam is 105 m high from foundation rock, 260 m long at the crest and the total earthfill is 2,500,000 $m^3$ (Fig. 13). A minimum safety factor of 1.37 was obtained as a result of studies made by the modified Fellenious' method for various load conditions. A horizontal seismic coefficient of 0.15 was taken in the design calculations taking into consideration the earthquake records taken at the damsite as well as the results of vibration experiments with a model. Acceleration meters and displacement meters are embedded in the dam and observations of earthquake and measurements of the movement of the dam are being taken.



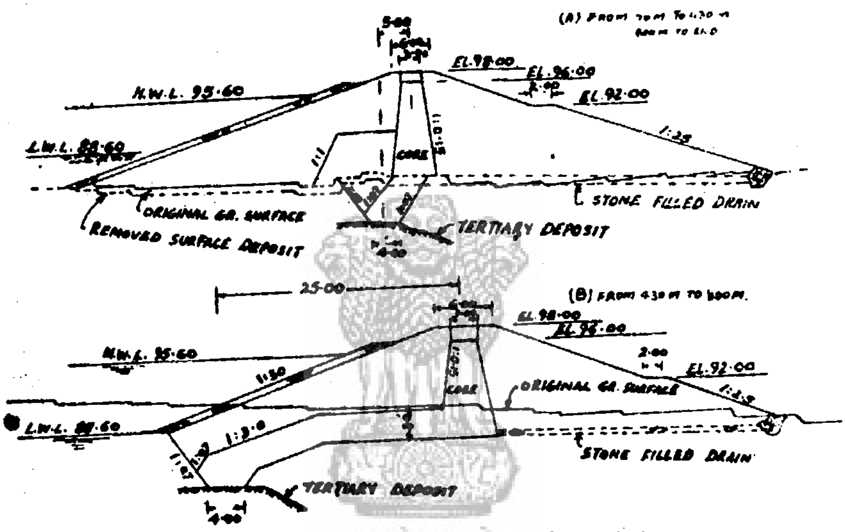

MAKIO DAM

FIG. 13 (APP. P)

(3) *Ojiya Dam—Japan*

Ojiya Dam is a rolled-fill earth structure with a central core located at the downstream of the Shinano River in the Hokuriku region. It is 954 m long at the crest, 18.8 m in maximum height above ground level and 29.3 m above Tertiary bed rock (Fig. 14). Calculations of the stability of the dam were carried out as follows:—

The stability of the dam against shear force was studied at the base and on the surface of the Tertiary bed of the highest section under normal and earthquake conditions. At the same time the stability of the slopes of the dam under normal condition was examined by the slip-circle method when rapid drawdown took place on the upstream side and when there was a snow load of 1.75 t/m² working on the downstream slope and its seepage pressure was at full work.




OJIYA DAM - SECTION
FIG. 14
(APP. P)

As a result of the studies, the minimum safety factors of 1.18 at the base of the dam, 1.04 on the surface of the Tertiary bed in time of earthquake shock and 1.48 on the upstream slope and 1.47 on the downstream slope under normal condition were obtained. The angles of internal friction of the soil used in the calculation were determined by laboratory shear tests. A seismic coefficient of $K_h$= 0.1 was used as an earthquake force and the angle of internal friction was reduced by applying Dr. Sano's method. As the unit weight of earth below the seepage line, the saturated unit weight when it acted as an external force and the bouyed unit weight when it acted as resistance were adopted. The excessive pore pressure in the clay core taken in the calculation of the slope stability was 1.25 times the vertical height of the core from the point under consideration.

(4) *Santa Clara Water Conservation District Dams—U.S.A.*

Five rolled-fill earth dams were built by the Santa Clara Water Conservation District in Santa Clara Valley, California, with special consideration of the seismic risk involved. Four of the dams are located in the area between the San Andreas and Hayward faults. These four dams have crest widths of 20 feet and 2½ : 1 slopes on each side. The outer slopes were of very pervious dumped materials. The core was rolled with sheepsfoot rollers at 325 psi. The upstream slopes were smoothed with a 12-inch layer of clean gravel and were faced with 4-inch thick by 16-foot square concrete slabs.

Coyote Dam, the fifth in Santa Clara Valley, is situated on the Coyote River, 25 miles southeast of San Jose, California. Since the dam is poised over the city of San Jose, it was designed for maximum seismic stability. Coyote Dam is 140 feet high at maximum section, is 900 feet long, and stores 40,000 acre feet of water. The dam is bisected by the trace of the Hayward fault. The maximum section of the dam is shown in Fig. 15.

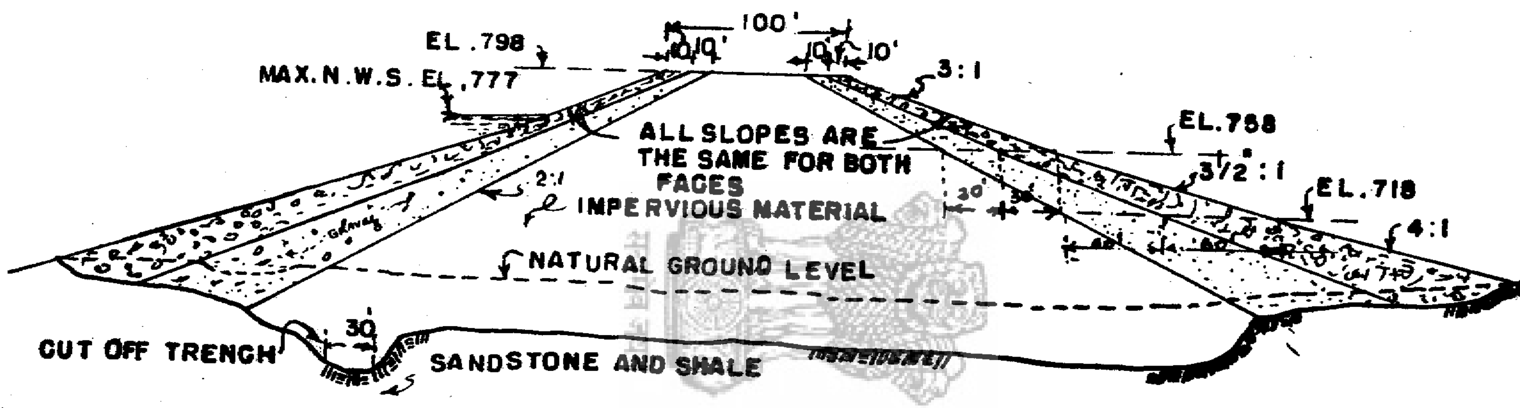
Since minor movement is recorded each year, and major movement is recorded every 30 to 40 years along the Hayward fault, design features for Coyote Dam include :

1. Spillway location was selected to provide a straight line path for flood waters in order to achieve the greatest support for the concrete lining and to provide the best rock material for the fill.
2. The outlet works are steel pipes placed in concrete filled trenches in the foundation. Valves are used on both ends.
3. The impervious core is designed to resist rupture by eliminating the concrete core wall and concrete facing on the upstream slope.
4. The cut-off trench is carried deep into the foundation so that cracks in the foundation will not allow water to pass under the dam.
5. The dimensions of the impervious section are such that the section will not be offset enough to let water through.
6. To prevent slumping, the impervious section is weighted down by coarse pervious material.
7. Extra heavy outer slopes were provided to close any tension fissures developed by stretching the dam.
8. Freeboard of 21 feet or three times the maximum vertical displacement of the fault. was provided to prevent overtopping if a major earthquake, maximum flood, and maximum wind occurred simultaneously.

(5) *Gyobyu Dam—Burma*

Gyobyu Dam on the Gyobyu Chaung, 55 miles north of Rangoon, Burma, is designed to resist seismic shock. Maximum height above streambed is 134 feet, length is 700 feet, and volume of fill is 370,000 cubic yards. This structure was completed in 1948. The interesting features of the design are :

1. A concrete core wall separated into panels by bituminous sheeting. The panels are 25 feet by 20 feet in area, and




COYOTE DAM

THE TRACE OF THE HAYWARD FAULT BISECTS COYOTE DAM NORMAL TO THE AXIS

FIG. 15 (APP. P)

7 feet 6 inches thick at the base to 4 feet thick at the top of the panel.

2. A 4'-6" wave-wall on the crest of the dam.
3. A downstream retaining wall backed by laterite fill.

(6) *San Andreas Dam—U.S.A.*

Two earth dams along the San Andreas fault withstood the 1906 earthquake with little damage to the structures. These, Crystal Springs Upper Dam and San Andreas Dam, are rolled earth dams with puddle clay core. Both were completed prior to 1900, and it is not known whether special consideration was given to the seismic stability of the structures. However, both dams have relatively flat slopes at 2:1, 3:1, 3½:1, and 3½:1 ratios. The Crystal Springs Upper Dam was not being used to form a reservoir and had water acting on both faces. San Andreas Dam is 95 feet above streambed, and 130 feet above the cut-off trench. It is 970 feet long (Fig. 16). The crest was raised 4 feet in 1928. The upstream slope is 3½ : 1 and the downstream 3 : 1. The cut-off trench 20 feet wide, extends to a minimum depth of 45 feet into the rock foundation.

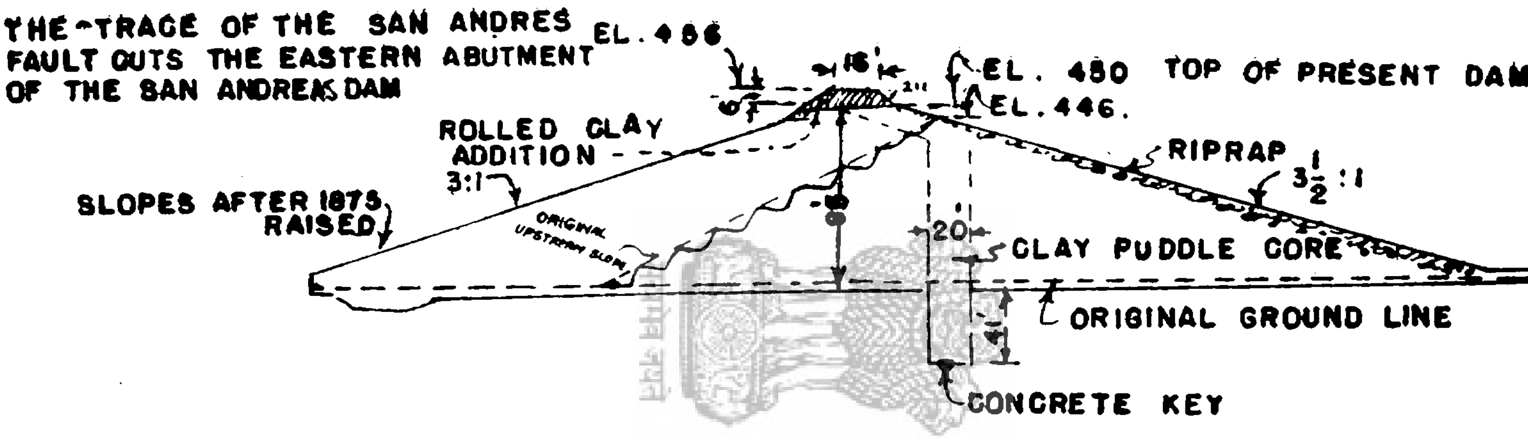
After the 1906 earthquake, San Andreas Dam and appurtenant structures were thoroughly inspected. The outlet tunnel around the east abutment was ruptured in the plane of the fault and the west section of the tunnel moved 8 feet north. The intake structure and associated pipes were demolished. Considerable settlement was noted. The dam proper had longitudinal cracks on the slopes and larger cracks normal to the axis. No observations were made regarding the effect of reservoir thrust. No leaks occurred and the structure is still used. It is to be noted that the design of San Andreas Dam, is very similar to Coyote Dam.

The San Andreas fault passes through the east abutment of the dam normal to the axis, but does not intersect the body of the dam.

**8. Summary of Experience in U.S.A. and Japan**

From a study of the examples of earth and rockfill dams of Japan and U.S.A. quoted above, the following points are brought out which are noteworthy :—

1. In all the cases only horizontal earthquake acceleration is taken into account.
2. Sudden drawdown does not appear to have been combined with earthquake condition.
3. Increase in pore pressure does not appear to have been considered.
4. The value of cohesion is not altered under earthquake condition.
5. In the case of Ojiya Dam, the angle of internal friction is reduced under earthquake loading.
6. Under earthquake conditions, the factor of safety is just over unity.




60 0 60

SCALE OF FEET

SAN ANDREAS DAM

FIG. 16 (APP. P)

(FROM TECHNICAL MEMORANDUM 641 OF U.S.S.R.)

**9. Procedure for Earthquake Resistant Design of Dams in Turkey**

The stability analysis of dams is made in the following cases :—

1. Earthquake during construction.
2. Earthquake at full reservoir.
3. Earthquake at instantaneous drawdown of reservoir.

In the first degree earthquake regions, horizontal earthquake acceleration of 0.20 g and vertical acceleration of $1/3 \times 0.2$ g are taken into account and safety factors corresponding to these two cases are computed.

In the second degree earthquake regions, horizontal earthquake load at the value of 9.1 g and vertical earthquake load at the value of $1/3 \times 0.1$ g are taken into account and safety factor corresponding to those two cases are computed.

In the case of earthquake, the safety factor is not less than unity. The hydro-dynamic effects created by reservoir water during the earthquake are not taken into consideration for earth dams. The stability analysis is made with the modified Swedish slip-circle method. In that analysis the effects of the above mentioned earthquake loads are considered.

Because of lack of seismographic survey, the features of the earthquake waves and the characteristics of the materials are not known and as such, the resonance analysis of the embankments is not made.

In the earth dam located in earthquake regions, the following conditions are taken into account :—

1. As during an earthquake, cohesionless material is likely to become unstable, the volume of such material is kept at the minimum.
2. The volume of impervious core is increased to prevent the leakage of water through the fissures which might be created by the earthquake.
3. The camber at the crest of the dam is increased in the earthquake regions against the overtopping of water which might result by settlements due to earthquakes.
4. The rocks on the surface of ground are cracked or will be cracked due to earthquakes in the earthquake regions. Therefore the cut-off trench which is going to be excavated in the rock is taken even deeper to prevent the seepage of water (to 15 m. depth at places close to the faults).

## ANNEXURE I TO APPENDIX P

# FORMULAE FOR DETERMINING HYDRO-DYNAMIC PRESSURE

Formulae used for determining the hydro-dynamic pressure exerted on vertical and sloping faces of the dam by horizontal earthquake are as follow :—

**A. Horizontal Earthquake**

1. For dam with vertical or sloping upstream faces, the variation of hydro-dynamic earthquake pressure with depth is given by the equation :—

$$Pe = c\ \alpha\ wh$$

and $C = \frac{Cm}{2} [y/h\ (2\quad y/h) + \sqrt{y/h\ (2-y/h)}\ ]$

**where,**

Pe=pressure normal to the face,

=earthquake intensity,

$\alpha = \frac{\text{earthquake acceleration}}{\text{acceleration due to gravity}}$

w=unit weight of water,

h=maximum depth of reservoir,

y = vertical distance from the reservoir surface to the elevation in question, and

Cm=a dimension less pressure coefficient.

2. For dams with combination of vertical and sloping face :

(a) If the height of the vertical portion of the upstream face of the dam is equal to or greater than one half the total height of dam, analysis as if vertical throughout.

(b) If the height of the vertical portion of the upstream face of the dam is less than one half the total height of the dam, use the pressures on the sloping line connecting the point of intersection of the upstream face of the dam and the reservoir surface with the point of intersection of the upstream face of the dam with the foundation.

**B. Vertical Earthquake**

1. The component of water pressure normal to the face of the dam should be modified by an appropriate acceleration factor.

2. The unit weight of the concrete should be modified by the same acceleration factor.

An earthquake acceleration up to approximately 0.3 g is only about one half as effective in silt or soil masses as it is in water. Since the unit weight of water is also approximately on half that of silt, it is sufficient to determine the increase in silt pressure due to earthquake as if the water is extended to the base of the dam. This increase is then added to the static silt pressure.

**C. Load Combinations**

Standard load combinations :

(i) Normal water-surface elevation, ice and silt (if applicable), and normal uplift.

(ii) Normal water-surface elevation, earthquake, silt (if applicable), and normal uplift.

(iii) Maximum flood-water surface elevation, silt (if applicable), and normal uplift.

(iv) Extreme load combination (gravity dams only) : Maximum flood water elevation, silt (if applicable), and extreme uplift (drains inoperative),

(v) Reservoir empty :

The condition of empty reservoir (without earthquake) should be computed for reinforcement design, grouting studies, or other purposes.

Combinations of transient loads, each of which has only remote probability of occurrence at any time, cannot be considered as reasonable basis for design. For example, maximum earthquake should not be combined with maxmum design flood.

**D. Shear Factor of Safety**

The expression used is :

$$Q=\frac{CA+N\ \text{Tan}\ \phi}{H}$$

where

C=cohesion,

A=area of the base considered,

H=Summation of shear forces,

N=Summation of normal forces, and

Tan $\phi$=coefficient of internal friction.

For gravity dams, the shear friction factor of safety should not be less than four for standard loading A, B or C and should be sufficient to ensure stability when tested for extreme loading condition. D.

**(E) Quality and Strength of Concrete—Allowable Stresses**

Although concrete is known to possess some tensile strength, quantitative evaluations are uncertain and speculative. Tensile failures in the form of cracking may in some cases have a very important effect on the structural behaviour of a dam; in other cases they may be unimportant. Tensile stresses due to twist action and temperature changes probably exist in all concrete gravity and arch dam. How important tensile stresses are in individual cases must be settled by considering the location, magnitude and direction of the stresses, and the effects of cracking on the behaviour of the structure.

Concrete strength should be determined by compressing to failure 18×36 inch cylinders of the full mass mix which have been cured in sealed containers at temperatures approximating those expected in the structure. These strengths should be such that 80% of all combined list values exceed the strength required.

The compressive strength of concrete determined as here-to-fore specified should satisfy early load and construction requirements and at 365 days' are should have a ratio to the allowable working stress as determined by the designer. However, in no case should the ratio be less than four, nor the allowable working stress exceed 1000 psi.

*APPENDIX—Q*

## ESTIMATE FOR EXPLOSION EXPERIMENTS PROPOSED TO BE CONDUCTED AT DAMSITES

| Item No. | Sub-heads and items of work | Quantity or No. | Rate | Per | Amount in. Rs | Total |
|---|---|---|---|---|---|---|
| | | | Rs. nP. | | | |
| 1. | Drilling 10 Nos. of holes 2⅜" dia. and average depth 100 ft. (the depth will vary from 50′ to 250′). | 1000 rft | 30·00 | ft | 30,000·00 | |
| 2. | Gelatine .. .. .. | 1500 lbs | 3·00 | lb | 4,500·00 | |
| 3. | Electronic equipment for testing purposes | | Lump sum | | 3,000·00 | |
| 4. | Miscellaneous expenditure including staff and equipment to be arranged from CWPRS for conducting the experiments. | | Lump sum | | 3,000·00 | |
| | TOTAL .. | | | | 40,500·00 | |
| | Add 3 % contingencies .. | | | | 1,215·00 | |
| | TOTAL .. | | | | 41,715·00 | |

*APPENDIX—R*

## VALUES OF SEISMIC COEFFICIENTS USED IN THE DESIGNS OF DAMS

| Sl. No. | Name of Dam | Zone according to ISI Classification | Type of Dam | Height | Horizontal acceleration | Vertical Acceleration |
|---|---|---|---|---|---|---|
| 1. | Bhakra .. .. .. | IV | Gravity | 740 | 0·15 | 0·075 |
| 2. | Barapani .. .. .. | VI | „ | 236 | 0·25 | 0·10 |
| 3. | Pong (Beas) .. .. | IV | „ | 210 | Res. empty 0·10 | 0·10 |
| | | | | | Res. full 0·20 | 0·10 |
| 4. | Kosi .. .. .. | V | Earth | 37 | 0·20 | 0·10 |
| 5. | Nalkari (Bihar) .. | IV | „ | 122 | 0·10 | 0·05 |
| 6. | Kopili .. .. .. | VI | „ | 106 | 0·22 | 0·11 |
| 7. | Ramganga .. .. | IV | Rockfill | 412 | 0·20 | 0·10 |

**Note:**

In Indian Standard Recommendations for Earthquake Resistant Design of Structures—IS: 1893-1962 issued by the I.S.I. in November 1962, the following values of horizontal acceleration have been given zone-wise (paras 3.4.1 & 7.1)

| Zone No. | Horizontal Seismic Coefficient | | |
|---|---|---|---|
| | Hard Soil | Average Soil | Soft Soil |
| VI | 0·16 | 0·20 | 0·24 |
| V | 0·12 | 0·16 | 0·20 |
| IV | 0·10 | 0·12 | 0·16 |
| III | 0·08 | 0·10 | 0·12 |
| II | 0·04 | 0·06 | 0·08 |
| I | 0 | 0·02 | 0·04 |
| 0 | 0 | 0 | 0 |

(The vertical seismic coefficient to be assumed as half the horizontal coefficient.)

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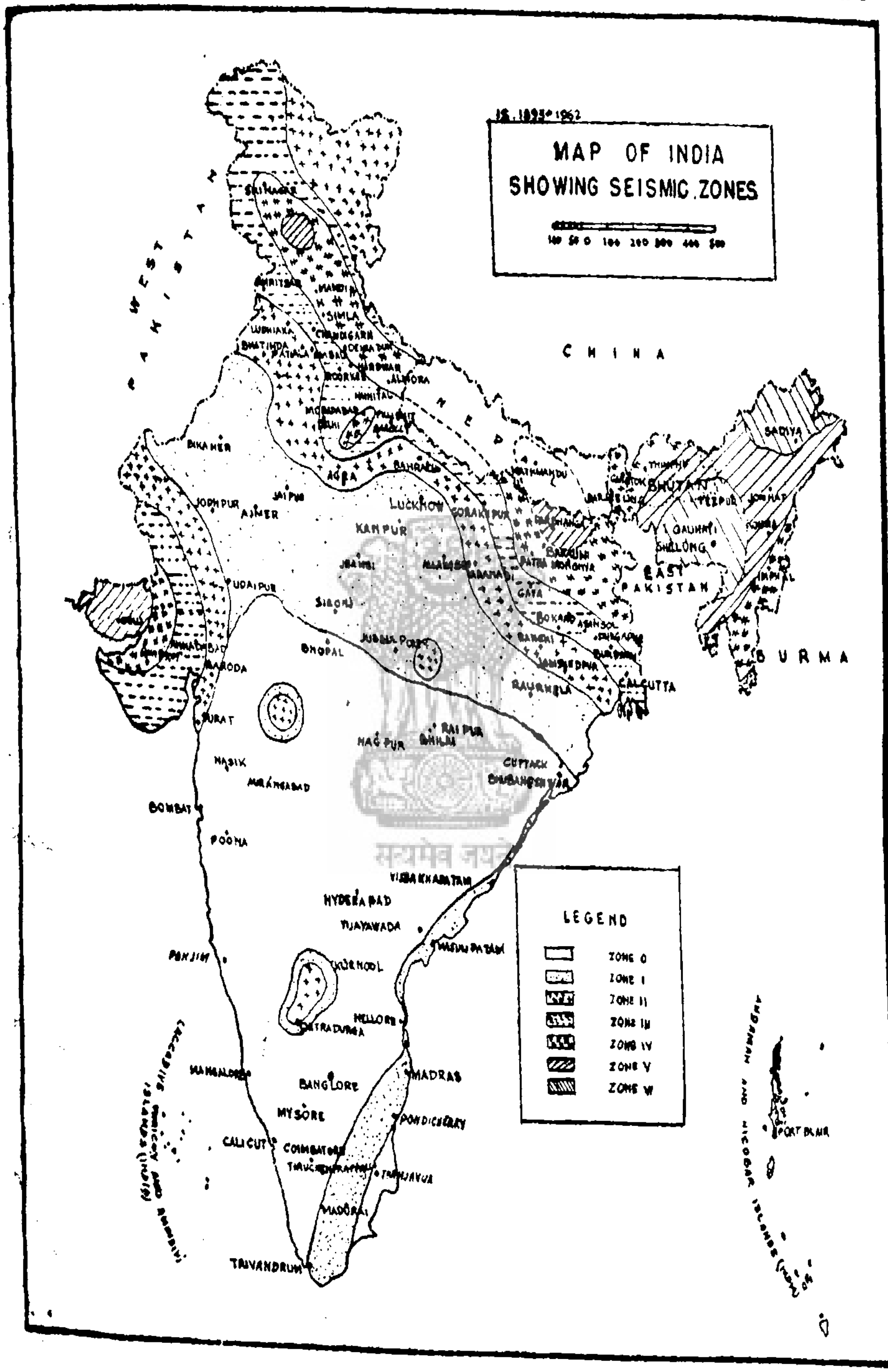
सत्यमेव जयते


IS. 1893-1962
MAP OF INDIA
SHOWING SEISMIC ZONES
LEGEND
ZONE 0
ZONE I
ZONE II
ZONE III
ZONE IV
ZONE V
CHINA
BURMA
EAST PAKISTAN
BHUTAN
SRINAGAR
AMRITSAR
SIMLA
CHANDIGARH
BHATINDA
ALMORA
BIKANER
JODHPUR
AJMER
JAIPUR
AGRA
LUCKNOW
KANPUR
UDAIPUR
BHOPAL
BARODA
SURAT
NASIK
BOMBAY
POONA
NAGPUR
RAIPUR
CUTTACK
CALCUTTA
GAYA
RANCHI
SHILLONG
SADIYA
HYDERABAD
VIJAYAWADA
KURNOOL
PANJIM
NELLORE
MANGALORE
BANGLORE
MADRAS
MYSORE
PONDICHERRY
CALICUT
COIMBATORE
MADURAI
TRIVANDRUM
PORT BLAIR