

**REPORT
OF
THE COMMITTEE ON INSTRUMENTATION
FOR
DAMS & HYDRAULIC STRUCTURES**

PART I & II



**MINISTRY OF IRRIGATION AND POWER
GOVERNMENT OF INDIA
NEW DELHI**

December, 1965

C.W. & P.C. No. 164

15th April 1968

(ii)

The Report is in two parts.

Part I is a general report dealing with the specific terms of reference embodied in the Government Resolution.

Part II (Separate Volume) is a detailed report describing necessity for instrumentation of important structures, the objectives to be sought, the nature of the measurements to be made and a description of the various instruments generally adopted for such measurements.

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BHAKRA & BEAS DESIGNS ORGANIZATION, KAKA NAGAR, NEW DELHI-11

No. 1044-47/CDE

December 28, 1965.

To

The Secretary to the Govt. of India,
Ministry of Irrigation & Power,
New Delhi-1.

SUBJECT :—*Final Report of the Committee on Instrumentation of Dams & Structures.*

Sir,

Pursuant to Resolution No. 20(31)/63-Policy, dated October 31, 1964, of the Government of India in the Ministry of Irrigation & Power, a Committee was constituted for the study of instrumentation of dams and structures in its various aspects. I have the pleasure to submit herewith the final report of this Committee.

The Report is in two parts. Part I is a general report dealing with the specific terms of reference embodied in the Government Resolution. Part II is a detailed statement of the necessity for instrumentation of important structures, the objectives to be sought, the nature of measurements to be made and a description of the various instruments generally adopted for such measurements. The Committee has felt it desirable to add Part II of the Report so as to provide the background in the context of which Part I of the Report has been written.

It is hoped that this Report will not only serve its direct purpose, *i.e.* to examine instrumentation in all its aspects including a review of the instrumentation already done in the country and to recommend the best ways and means for the effective implementation of measurement programmes in our hydraulic structures but will also be a useful technical reference for those who might plan similar programmes in the future.

Yours faithfully,

(P. S. BHATNAGAR)

Chairman,

Committee on Instrumentation of Dams & Structures.

Encl : Report in two parts.

Copy with enclosures to :

- (i) Chairman, Central Water & Power Commission, New Delhi.
- (ii) Member (Design & Research), Central Water & Power Commission.
- (iii) General Manager, Beas Project, Talwara.
- (iv) General Manager, Bhakra Dam, Nangal.
- (v) All Members of the Committee.

(ix)

INTRODUCTION

1. A Committee was constituted by the Government of India in the Ministry of Irrigation and Power by Resolution No. 20(31)/63-Policy, dated 31st October, 1964 (Appendix 0.01) for a study of instrumentation in various dams and hydraulic structures after visits to dams where necessary. The Resolution laid down the following terms of reference :—

- (i) To make a general review of the instrumentation done so far in various dams in the country including general examination of data and results obtained therefrom and their interpretation as indicative of structural action ;
- (ii) To assess future requirements of instruments upto the end of Fourth Plan, with particular reference to the requests of the projects now under construction ;
- (iii) To examine different types (such as Carlson elastic wire type, vibrating wire type etc.) of instruments and recommend the best suited for our projects ;
- (iv) To recommend the best arrangement for recording, storage, analysis and interpretation of field readings from instruments ;
- (v) To recommend the most suitable method of procurement of instruments for various State projects in the absence of indigenous products ;
- (vi) To examine the need for maintaining a stock of such instruments in the country, so that the requirements of projects can be properly met and instrumentation receives due implementation without any setback to the construction programme ; and
- (vii) To formulate a programme for the progressive manufacture of these instruments indigenously.

2. The Committee comprised the following members :—

- (i) Shri P. S. Bhatnagar,
Chief Design Engineer,
Bhakra & Beas Designs Organization,
Kaka Nagar, New Delhi. *Chairman.*
- (ii) Shri K. R. Datye,
Superintending Engineer,
Deccan Irrigation Circle III,
Nasik. *Member*
- (iii) Shri B. Pant,
Chief Research Officer,
Central Water & Power Research Station,
Poona. *Member*

- (iv) Shri I. P. Kapila,
Director Designs,
Bhakra & Beas Designs Organization,
Kaka Nagar, New Delhi. *Member*
- (v) Shri V. K. Sondhi,
Deputy Director,
Central Water & Power Commission.
Member-Secretary.

3. The Committee held the following meetings :—

Date of meeting	Place of meeting	Members attending
Dec. 15, 1964	Instrumentation Laboratory, C.W.&P.R.S., Poona.	Shri P. S. Bhatnagar Shri K. R. Datye Shri B. Pant Shri V. K. Sondhi.
Jan. 28, 1965	Gandhi Sagar Dam, Circuit House.	Shri I. P. Kapila Shri K. R. Datye Shri B. Pant Shri V. K. Sondhi.
March 2, 1965	Koyna Dam, Rest House, Koyna.	Shri P. S. Bhatnagar Shri I. P. Kapila Shri K. R. Datye Shri B. Pant Shri V. K. Sondhi.
April 27, 1965	Bhakra & Beas Designs Organisation, N. Delhi.	Shri I. P. Kapila Shri K. R. Datye Shri B. Pant Shri V. K. Sondhi.
July 16 & 17, 1965	Instrumentation Lab. C.W. & P.R.S., Poona.	Shri I. P. Kapila Shri K. R. Datye Shri B. Pant Shri V. K. Sondhi.
August 6 & 7, 1965.	Bhakra & Beas Designs Organization, N. Delhi.	Shri P. S. Bhatnagar Shri I. P. Kapila Shri K. R. Datye Shri B. Pant Shri V. K. Sondhi.
August 24 & 25, 1965.	Office of S.E., Deccan, Irrigation Circle III, Nasik.	Shri I. P. Kapila Shri K. R. Datye Shri B. Pant Shri V. K. Sondhi.
Oct. 20 & 21, 1965.	Bhakra & Beas Designs Organization, N. Delhi.	Shri P. S. Bhatnagar Shri I. P. Kapila Shri K. R. Datye Shri B. Pant Shri V. K. Sondhi
Nov. 9 & 10, 1965.	Inspection House, Maharashtra Govt., Poona.	Shri P. S. Bhatnagar Shri K. R. Datye Shri B. Pant Shri V. K. Sondhi.

REPORT OF THE COMMITTEE ON INSTRUMENTATION FOR DAMS AND HYDRAULIC STRUCTURES

4. The Members of the Committee visited the following projects :—

- (a) Gandhi Sagar (visited by I. P. Kapila, K. R. Datye, B. Pant & V. K. Sondhi on 28th/29th Jan. 1965).
- (b) Ranapartap Sagar Dam under construction (visited by I. P. Kapila, K. R. Datye, B. Pant & V. K. Sondhi on 28th/29th Jan. 1965).
- (c) Kotah Barrage (visited by I. P. Kapila, K. R. Datye, B. Pant & V. K. Sondhi on 28th/29th Jan. 1965).
- (d) Kunnu Syphon Aqueduct (visited by K. R. Datye and B. Pant on 30th January, 1965).
- (e) Koyna Dam & Power Plant (visited by all members on March 2, 1965).
- (f) Sharavathy Project (visited by P. S. Bhatnagar, I. P. Kapila, and V. K. Sondhi on 5th March 1965, and K. R. Datye on 31-10-1965 to 2-11-1965).
- (g) Bhakra Dam (visited by K. R. Datye, B. Pant and V. K. Sondhi on 29th/30th April, 1965).
- (h) Hirakud Dam (visited by K. R. Datye on 24th/25th May, 1965).
- (i) Rihand Dam (visited by V. K. Sondhi on 29th to 31st July, 1965).
- (j) Tungabhadra Dam (visited by V. K. Sondhi on 10th/11th August, 1965).

In addition, the Committee had the advantage of first hand information with Shri K. R. Datye regarding Gangapur, Bor and Girna Dams in Maharashtra where instrumentation has been carried out.

5. Authorities concerned with the River Valley Projects in the Centre and various States were requested to supply particulars of instrumentation affected at projects under their control. This information was sought in two proforma statements (Appendix 0.02). Replies from 46 authorities were received, out of which 27 indicated that either no structure was under their charge or no instrumentation was provided, and 19 (covering 30 structures) supplied some information. The information as supplied or as collected is in respect of the following structures :—

- (a) *Concrete Dams*
Bhakra Dam
Koyna Dam
Rihand Dam
- (b) *Masonry Dams*
Gandhi Sagar Dam
Hirakud Dam
Linganamakki Dam

Malampuzha Dam
Mani Muthar Dam
Nagarjunasagar Dam
Parambikulam Aliyar Project
Tungabhadra Dam
Walayar Dam

- (c) *Earth Dams*
Bor Dam
Buggavagu Dam
Dantiwada Dam
Datla Tank
Gangapur Dam
Ghod Dam
Girna Dam
Hathmati Dam
Hirakud Dam
Jirgo Dam
Kotah Barrage
Linganamakki Dam
Meshwa Dam
Nalganga Dam
Nanak Sagar Dam
Sampana Dam
Ukai Dam

- (d) *Other Structures*
Bhira Tunnel
Koyna Head Race Tunnel
Koyna Power House Foundation
Koyna Pressure Shaft
Kunnu Syphon Aqueduct
Nagarjunasagar Right Canal Tunnel
Sharavathy Water Conductor System
Vir Dam.

The information is consolidated at Appendix 0.03.

6. As a result of detailed study of the information pertaining to instrumentation at various projects, discussions with project engineers, information obtained from persons outside India who are actively engaged in this work, discussion with representatives of manufacturers of instruments, and of the published literature on this subject, the Report of the Committee is submitted herewith. This report is in two parts. Part I is the general report dealing with the specific terms of reference embodied in the Resolution of the Government of India. Part II is the detailed report which deals with objectives of instrumentation, nature of measurements and their purpose and description of the various instruments generally adopted for such measurements. Detailed information pertaining to principles of working and construction, considerations governing location and layout, method of installation, observation and analysis of observed data has either

INTRODUCTION

been included or reference has been made to relevant technical publications. The Committee has felt it desirable to add Part II of the Report so as to provide the background in the context of which Part I of the Report has been written.

The Committee has not examined any measurements relating to stability of slopes within the reservoir area which affect the safety and utility of a structure, as this was outside the scope of the terms of reference. Measurements relating to meteorological data, and seismic characteristics of dams and structures have likewise not been examined as being outside the scope of the terms of reference of the Committee, except for the

brief mention wherever considered necessary in this report.

7. The work of the Committee was seriously handicapped by the fact that the Chairman was hospitalised for a period of four months from April to the end of July 1965. On this account the period for the submission of the Committee Report was first extended up to the end of September 1965 and subsequently up to the end of Dec. 1965 by Government of India, Ministry of Irrigation & Power Resolution No. 20(31)/63-Policy, dated 6th July, 1965 (Appendix 0.04) and Resolution No. 20(31), dated 29th September, 1965 (Appendix 0.05) respectively.

ACKNOWLEDGEMENTS

The Committee is specially indebted to Shri R. P. Sharma, Senior Design Engineer of Bhakra & Beas Designs Organization, New Delhi for his valuable contribution in the deliberations of the Committee and in compilation and editing the Committee Report.

The following officers also assisted in the work of the Committee:

Shri V. A. Parkash, Deputy Director and Shri N. C. Gupta, Asstt. Design Engineer, both of Bhakra & Beas Designs Organization, New Delhi; Shri Shiyekar, Asstt. Engineer of Deccan Irrigation Circle, Nasik; Shri A. S. Khot, Research Officer, Central Water & Power Research Station, Poona; Shri K. M. Nagbhushan, Asstt. Director and Shri R. N. Poddar, Asstt. Director, both of Central Water & Power Commission.

Typing of the final Report was done by Shri O. P. Gupta, Stenographer, Bhakra & Beas Designs Organization, New Delhi.

PART I

GENERAL REPORT

1.0 TERM OF REFERENCE—I

“To make a general review of the instrumentation done so far in various dams in the country including general examination of data and results obtained therefrom and their interpretation as indicative of structural action.”

For the purpose of the general review required by this term of reference, the Committee considered it appropriate to cover the instrumentation provided at the following projects :

(a) *Concrete Dams*

Bhakra Dam
Koyna Dam
Rihand Dam

(b) *Masonry Dams*

Gandhi Sagar Dam
Hirakud Dam
Linganamakki Dam
Nagarjunasagar Dam
Tungabhadra Dam
Mani Muthar Dam

(c) *Earth Dams*

Bor Dam
Gangapur Dam
Girna Dam
Hirakud Dam
Jirgo Dam
Kotah Barrage
Nanak Sagar Dam

(d) *Other Structures*

Koyna Powerhouse Generator Floor
Koyna Pressure Shaft
Kunnu Syphon Aqueduct
Nagarjunasagar Right Canal Tunnel

1.1. GENERAL REVIEW

In order to carry out a general review of the instrumentation done so far in various dams in the country, the Committee has sought answers to the following questions :—

- (a) Has instrumentation been included at all important structures in the country?
- (b) Where instrumentation has been provided, has it been properly conceived both in respect

of the measurements to be taken and in respect of the means for such measurements?

- (c) Have instruments been properly installed to ensure a reasonable period for which data would be available?
- (d) Has a proper record been maintained of measurement data and has the data been adequately and effectively processed and interpreted?
- (e) Has the measurement data provided useful information for assessing structural action?

1.1.1. A tabulation of all the information supplied by respective projects is at Appendix 0.03. This covers type of measurements, the instruments used, and status of observations and processing of data. The tabulation is presented separately for different types of dams, and includes a separate statement in respect of other relative structures. There are some notable dams and structures which do not appear in these tabulation and the Committee understands from the response to the proforma questionnaire that no instrumentation was done at these projects. As construction at such projects was undertaken within the last 15 years or so, at a time when the importance of instrumentation and post-construction assessment of structural action had been generally recognised, the omission of this facility in the planning stage or thereafter is indicative of a general lack of appreciation of the need for such measures being incorporated at appropriate time. Dams, wherever they are constructed, constitute the key element in any river development programme, and it should be obligatory on the part of the agency responsible that it has means available for making the best possible appraisal of the magnitude of the forces developed and the manner in which the structure is responding by providing for at least essential measurements.

1.1.2. Regarding those projects where instruments have been provided, arrangements for measurement of deflection, joint opening, stress, strain, temperature, pore pressure and uplift have been made, but all these are not available at each project. None of the projects included geodetic measurements of displacement in the original plans, which is unfortunate as this represents a valuable aid for obtaining information regarding the integrated behaviour of the dam, abutments, and foundation. A review of the instrumentation provided indicates that whereas generally it is adequate, in many cases better and more useful

REPORT OF THE COMMITTEE ON INSTRUMENTATION FOR DAMS AND HYDRAULIC STRUCTURES

information could have been obtained with greater attention at the planning stage. For example:—

(a) by providing targets and bench marks and instrument stations at suitable locations at the dam site prior to commencement of dam construction, measurements of overall displacement of dam and foundation by geodetic methods could be started with the commencement of dam construction, enabling assessment of integrated behaviour of the structure;

(b) by installing reverse (inverted) plumb lines at appropriate locations, displacement of reference points in plumb line observations resulting from foundation deformations could be measured enabling determination of overall dam deflections from plumb line observations, and more realistic comparison of the actual and theoretically determined deflection values;

(c) by providing metal tags, bearing the instrument name and number near the cable lead terminations of respective instruments before commencement of their embedment, identification of these cable terminations could be established for correct correlation of calibration and observed data with respective meters;

(d) by determining the creep characteristics of the concrete of respective dams, in advance of the start of the instrument installation, stress evaluations from observed strainmeter data could become available soon after observations, enabling assessment of structural behaviour of the dam and foundation, as construction progressed;

(e) by providing a transverse foundation gallery in the respective dam blocks selected for uplift pressure measurements vertical uplift pipes could have been provided for regular and continuous uplift pressure observations, as reaming or redrilling through these pipes could be undertaken when necessary to ensure their proper functioning;

(f) by providing porous tube piezometers in sufficient numbers the shape of the phreatic line would have been more definitely ascertained;

(g) by providing stand-pipes in sufficient numbers on the upstream and downstream of grout curtain and by taking standpipes to a depth sufficient to penetrate the pervious strata more definite indication of the curtain performance could have been obtained;

(h) by collecting consolidation data on the soils in the vicinity of cross-arm installation comparison could have been made of the observed and computed settlement;

(i) by adequately training concerned personnel in the procedures for testing calibration, installation and observation of instruments and analysis of observed data sufficiently in advance of the commencement of instrument embedment, larger volume of reliable data could become available and more realistic assessment of structural performance expedited.

1.1.3. An appraisal of the installation procedures is possible only from the number of instruments reported

inoperative or unreliable. Post-installation precautions if not properly observed can also be responsible for unreliable readings. These include proper care of cables at the gallery faces. At some projects, cable leads were not terminated at boards and were hanging from the gallery face. Proper contacts with measuring test set in such cases is not ensured and moisture has easy access to the cable, rendering readings being questionable. At one project, care had not been taken to maintain a record of the indentifying particulars, on account of which, calibration data of the concerned instruments was not known.

Information pertaining to the number of operative and inoperative instruments of the various types embedded in the projects included in this report is shown in the Tabulation (Appendix 0.03) para 1.1.1.

Even though instruments may be operative, reliability of the readings recorded on these may be affected on account of impaired grounding resistance, penetration of moisture into the cable leads, poor maintenance of test set and instrument terminal contacts on the terminal panels, instability of the standard testing lead used for instrument terminals and test set connection etc.

1.1.4. Generally, not much effort is evident for systematic record, processing and interpretation of data. As such, project authorities are, in all probability, not fully apprised of the behaviour of their structures. The Committee has observed that this situation arises out of the fact that personnel are not earmarked for this work specifically, that there is generally no continuity of personnel connected with the earlier stages of this work, and that there is no sufficient liaison with the available design and research personnel. It is necessary to emphasize that the primary utility of measurements is to the project itself for an assessment of the safety of the structure. The academic questions which may arise for a better understanding of structural phenomena for design of future projects, though an important aspect, must take a second place. Every effort should, therefore, be directed at each project to properly process data and arrive at logical and reasonable interpretations of results obtained (Refer letter dated Feb. 23, 1965 from Prof. J. M. Raphael, Appendix I-1.01).

1.2. GENERAL EXAMINATION OF DATA RESULTS AND INTERPRETATION

A. Concrete Dams

1. Displacements

(a) *Plumb Lines*.—Plumb lines have been installed at all the concrete dams under review. At Bhakra, there are 3 plumb lines, one in the maximum spillway cantilever, and one each in the abutment cantilevers on either side of the spillway.

TERM OF REFERENCE-I

Plumb wells for installation of these plumb lines are vertical formed (unlined) openings. Interconnection of the plumb well in one of the abutment blocks developed with the adjacent transverse contraction joint through a crack resulting in leakage into the well and consequent disturbance to the plumb line during observation. This situation suggests the necessity of steel lining the plumb line wells.

At Koyna Dam there are 2 plumb lines located in maximum spillway cantilever and in the abutment cantilever on right side of spillway.

At Rihand Dam there is only one plumb line, installed in one of the power intake blocks.

Deflection measurements of the plumb lines at Bhakra normal and parallel to the axis have been taken by a micrometer slide and microscope at a sufficient number of points along the lines. The data is being processed regularly, and is presented in the following form:—

- (i) Deflections of vertical cantilevers (See Fig. I-1.01)
- (ii) Variation of deflection with time (See Fig. I-1.02)
- (iii) Variation of deflection with reservoir level (See Fig. I-1.03).

However, the deflections obtained are not absolute values as initial measurements before imposition of water load were not taken. Furthermore, the measurements do not account for the movement of the foundation which serves as the reference point. Recent developments in plumb line observations elsewhere now include the installation of a reverse (inverted) plumb line whereby the displacement of the foundation is observed with respect to a point at great depth which can be assumed to be fixed for all practical purposes (See Part II, Chapter 3). The two deficiencies are common to all the three dams under review, although the latter point, being a recent development could not have been foreseen at the planning stage. Nevertheless, some measures of relative displacement between the low and full reservoir levels is possible with the means available, and a comparison with theoretical values is possible to some extent. At Bhakra Dam, the results show that actual deflections normal to the axis for reservoir level about 40 feet lower than full level (EL. 1680) are somewhat less than the calculated values (See Fig. I-1.04).

At Koyna Dam, measurements of the dam displacements in two directions, are made by means of Huggenburger Coördimeter.

For reasons described above relevant to Bhakra Dam Plumb line observations, measurements at Koyna Dam provide only relative deflection. Observations revealed high order of deflection, indicating that reservoir fluctuations have only minor effect on the total deflection, while temperature change exercises substantial influence on the magnitude of deflections.

Study of plumb line deflections observations indicate that for dams of moderate height, and small fluctuation of reservoir level, factors other than hydrostatic load, determine the deflections to a marked degree.

At Rihand Dam, only one measurement point near the bottom end of the line is available, and measurements of deflection normal and parallel to the axis are taken by the Huggenburger Kaleidoscope. Deflections of the point of suspension only are available, and an assessment of the deflected cantilever shape is not possible. Measurements at the other points on this line were not possible as galleries at intermediate levels were not provided. The data has been processed. A deflector has been rigidly fixed to the plumb line about 2 feet above the plumb bob to prevent water leakage in the plumb line well from dripping into the dashpot, and a cover with a hole of $1\frac{1}{2}$ inch diameter through which the line passes has been fixed above the dashpot itself. It is possible that the actual movement of a plumb line may be affected on account of these two factors. The maximum deflection reported to the Committee during a site visit is about 1 inch. However, a comparison with computed values was not available.

(b) *Geodetic Observations.*—No geodetic observations were originally included in the instrumentation plans of all the three dams under review. This is an important measurement for absolute movements of the dam, abutments and foundations when related to fixed distant point. Efforts to establish reliable means for observing changes in displacement under varying load conditions are now in hand at Bhakra and Koyna Dams. These include triangulation surveys of targets fixed on the face of the dam and on the abutments, traverse survey of marker points at the crest of the dam; and level survey of bench marks fixed inside galleries of the dam and power plant and on abutments upstream and downstream of the dam.

(c) *Joint Meters.*—Joint meters have been provided at Bhakra and Rihand Dams. At Bhakra these were installed primarily in connection with the cooling and joint grouting operations being undertaken. They were located at various elevations in three blocks, maximum spillway block, and one abutment block on each side of the spillway. Jointmeters were also located in the upstream heel clay-stone strut.

Data from jointmeters at Bhakra has been processed to the extent necessary to enable concrete cooling and joint grouting operations being undertaken. No indications of structural behaviour from this data has been sought except that Bhakra upstream strut jointmeters indicated valley adjustments under increasing load, although these interpretations were not conclusive. Data at Rihand has not been processed.

(d) *Foundation Deformation Measuring Devices.*—The original instrumentation plans for all three projects did not include any means for observing settlement of the dam foundation. Efforts are now being

made for installing one such device at Bhakra (See Part II, Chapter 3).

(e) *Tilt Measurement Devices*.—No means were provided in the original scheme at any of the three dams for measuring tilt. This measurement actually supplements information from plumb lines, and geodetic surveys so far as gravity dams are concerned. At Bhakra Dam a tilt meter was recently placed in one of the galleries to observe vertical tilt. The data obtained is being processed but interpretation from a single measurement point may not be conclusive.

2. Stresses

Stressmeters of Carlson type have been installed at all the three dams under review. At Bhakra these have been provided at the base of the maximum spillway block. At Koyna they are located near the base of the maximum spillway cantilever. At Rihand these have been placed at the base of the power intake block.

At both Bhakra and Rihand, cable leads from stressmeters are properly terminated at terminal boxes at gallery faces and generally dry conditions prevail. At Koyna Dam the cable leads have also been properly terminated at the terminal boxes.

Data has been collected at all three dams but it has been processed at Bhakra and Koyna Dams so far.

The validity of this data and the results obtained is subject to satisfactory behaviour of the instruments and the accuracy of test set for taking readings. For the former, the grounding resistance of the meters requires to be checked periodically. This has not been done as a matter of normal observational routine at any of the three projects. Regarding the latter, calibration of the test set from time to time is desirable. This has been done for the Bhakra test sets, the calibration being checked at the National Physical Laboratory. At Koyna checking of test set calibration has not been done so far. The Rihand test set is checked in the project workshop, but calibration as such has probably not been done. However, some doubts arose with regard to the reliability of the test set during the site visit, indicating the necessity for periodic check and calibration of this facility.

Where strainmeters have also been provided contiguous to stressmeters, a check on the results is possible. However, this check has not yet been exercised at any of the three projects. The results of stressmeters at Bhakra have shown stresses within permissible limits. However, the interpretation of structural action from these results is as yet inconclusive.

3. Strains

Strainmeters of Carlson type have been installed at all the three dams under review. At Bhakra these are provided at three elevations in the maximum spillway

block and at two elevations each of an abutment block on either side of the spillway. At Koyna these are provided mainly at the base of the maximum spillway cantilever. At Rihand strainmeters are located at the base of the power intake blocks, and at the base of a spillway block.

Whereas data has been collected at all projects, processing has not been started except at Bhakra, where final results are not yet available. The Committee understands that at Bhakra the data had been processed after including effects of autogenous growth in concrete, which was determined from the "no-stress" strainmeters. However, a review indicated that revised growth strain is of a lesser magnitude than assessed earlier and it could be omitted from stress computation. The revised processing is now in hand.

A pre-requisite for processing strainmeter data is the assessment of creep, growth, and thermal properties of concrete, as also the Poisson's ratio. These properties except creep have yet to be established at Koyna Dam and at Rihand Dam all the properties other than Poisson's ratio require to be assessed.

The validity of data collected is subject to the comment relating to stress measurements. In addition, there is the additional factor that unless initial readings of strainmeters are taken immediately after installation, in the data obtained subsequently the effect of concrete creep cannot be accounted for.

4. Pore Pressures

No pore pressure measurements have been included in any of the three projects under review. The Committee considers it desirable to include this measurement at selective locations for a better appraisal of seepage phenomena through concrete but does not feel it to be obligatory at all projects.

5. Temperature

Carlson type thermometers have been embedded at all the three dams under review, and are located at various points to provide information regarding the temperature distribution within the body of dam at the upstream and downstream faces. Whereas, data has been processed at all the three projects, isotherms have yet to be drawn except for Koyna Dam, and temperature effects on structural behaviour have yet to be established.

6. Uplift Pressure

Uplift pressure pipes have been installed at all the three projects under review. At Bhakra Dam pipes have been installed at eight transverse sections, with 50 pipes in all, out of which 33 are vertical and the remaining are led to the concerned observation gallery with bends and horizontal lengths. Each pipe is provided with independent means for pressure observation. Readings are taken regularly at fortnightly intervals by a fixed pressure gauge. Pipes in which water level is below the gallery floor are observed by means

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of a sounder. At Koyna Dam, observations tip for measurement of uplift have been installed for three monoliths. By May 1965, a few of the tips in each monoliths have become inoperative apparently due to clogging. Still the remaining tips provide useful indication of the uplift phenomena. The procedure for observation is generally satisfactory. Valves are kept closed and sufficient time is allowed to elapse after fixing pressure gauges to allow readings to stabilise, before recording observations. Clogging is verified by opening the valves and in cases where there is no outflow, the pipe is taken as closed. As most of the uplift measuring pipes follow an alignment which involves bends, it is not possible to clean the uplift pipes by reaming and re boring. In the absence of observations of the equalisation time soon after installation and subsequently at regular intervals, it is not possible to ascertain whether or not there is any tendency for clogging.

At Rihand Dam pipes have been installed at one transverse section, and all except one have bends and horizontal lengths before they are exposed at the longitudinal foundation gallery. All the 8 pipes are connected to a common header to which a pressure gauge is attached. Pressure in any particular pipe is observed by closing the stop cock control between the pipe and the header on each of the other seven pipes and after allowing the pressure to build up over a period of 30 days. Thus observations at each pipe are taken after an interval of about 8 months.

Data obtained at Bhakra Dam has been processed regularly, and is plotted on transverse sections of the dam to show both water level (or equivalent water level for pipes under pressure) and the percent of net head prevailing (See Fig. I-1.05). Time plots for individual pipes are also maintained (See Fig. I-1.06). Results are interpreted in the context of the known geological features and remedial means are taken wherever possible to relieve any tendencies for high uplift. Generally, the uplift is much lower than the value assumed in design. Holes in the foundation through the pipes are redrilled and deepened as a regular maintenance feature to ensure that pipes are functioning properly and depicting uplift conditions faithfully.

At the Koyna Dam, observations have been made continuously every fortnight since December 1962, in all uplift observation pipes. Data has been processed regularly, time plots are maintained for individual pipes and the observations plotted on the cross sections to compare the observed uplift with the design assumptions. Pipes which are dry, are not considered while interpreting the results. As the pipes involve bends, it has not been possible to rebore holes which have become clogged.

Data on uplift obtained at Rihand Dam since 1958 has shown zero pressure. According to the procedure adopted for the measurement this probably indicates

that water in the pipe is below the gallery floor level, and no means are possible for observing it. If pipes are choked, their restoration is not possible by redrilling of holes on account of the bends and horizontal lengths. Consequently under the circumstances the data obtained is not likely to be representative of uplift conditions.

7. Seepage Measurement

Seepage of water from foundation drainage holes and formed drains in the dam is measured at all the three projects under review. At Bhakra Dam the primary drainage curtain is located in the Main Foundation Gallery, which is extended into the abutment drainage tunnels. A second drainage curtain is located further downstream in the Supplementary Foundation Gallery, which also extends into drainage tunnels into the abutments. Drainage holes have also been provided in transverse galleries extending under the spillway apron. Both abutments are drained including holes along the toe of the dam. Formed drains in the dam intercept seepage from the upstream face along the entire length of the dam.

At the Koyna Dam, seepage from different portions of the drainage system is measured but it has not been possible to measure the total seepage as curing water and leakages from construction water supply lines get mixed up with the seepage flow. Because of the stage construction, it was perhaps difficult to prevent extraneous water getting mixed with the seepage flow. All the same it would have been helpful to verify the total of the seepage measured at the various points with the collected seepage measured at the entry into the sump.

At Rihand Dam, drainage holes are located in the longitudinal foundation gallery and formed holes are provided in the dam.

The total seepage at Bhakra Dam fluctuates with the reservoir level (See Fig. I-1.07). The maximum seepage observed so far is 140 gpm. However, a significant proportion of this is from an abutment drainage tunnel in the left abutment which has been assessed by the project authorities as being correlated to ground water rather than the reservoir. The Committee understands that the project authorities are nevertheless examining this phenomenon further to establish the source of the seepage and would take corrective measures as indicated. In the mean time, additional drainage of the left abutment from downstream holes has been provided.

At the Koyna dam, continuous weekly review is made of observed uplift pressure versus design assumptions. It is found that the uplift is well below the design values. The record, however, indicates that the grouting by itself was not sufficient to bring down the uplift and drainage appears to be relatively more effective.

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At Rihand Dam, measurements have indicated only nominal seepage from foundation drainage holes. However, about 350 gpm are assessed to be seeping through formed drains in the power intake blocks which calls for careful examination and correction.

The total seepage is not excessive for the contributing area and foundation rock at Bhakra Dam. A close watch on the fluctuations and the implementation of corrective measures where indicated are required to be continued especially in the first few years of reservoir filling.

8. Recommendations

From the general examination of data, results and interpretation of instrumentation for concrete dams, the Committee recommends as follows:—

(a) The Committee recommends that as a general rule means should be provided for observing behaviour of all concrete dams. At these completed projects where this has not been done, efforts should now be made to do so to the extent possible under the prevailing circumstances.

(b) The Committee considers it necessary that a distinction be made between those measurements that are related to an assessment of safety, and others which are warranted only by the magnitude and importance of a structure, or by other special conditions, and also where the agency responsible considers that information obtained would provide a better understanding of structural phenomena for application to future designs.

(c) The Committee recommends the following measurements which should be obligatory for all concrete dams:

- (i) Displacement which includes: those determined by suspended plumb lines; those determined by geodetic measurement where warranted by the importance of a structure; those determined by embedded resistance jointmeters at contraction joints where grouting is required to be done; those determined by embedded resistance jointmeters or by other surface measuring devices for observing relative displacement of monoliths, particularly where appreciable foundation movements are anticipated, in such cases foundation deformation and tilt measurements will also be obligatory.
- (ii) Temperature of the interior of a dam as determined by resistance thermometers.
- (iii) Uplift pressure at the base of the dam at a sufficient number of transverse sections as determined by uplift pressure pipes, with means provided for reaming and deepening holes into the foundation.
- (iv) Seepage into the dam and appearing downstream therefrom.

(d) The Committee recommends the following measurements as, not being obligatory for all concrete dams, being undertaken only where warranted by the special circumstances of a project:—

- (i) Rock and foundation deformation,
Tilt
- (ii) Stress
- (iii) Strain
- (iv) Pore pressure (as distinct from uplift pressure)
- (v) Dynamic characteristics for structures located in seismic areas.

The Committee considers that these measurements would be beneficial for high dams, for structures of unusual design, for structures where unusual or doubtful foundation conditions exist, for the verification of design criteria, and for effecting improvement in future designs. The Committee considers that where such measurements are undertaken, a well planned and continuous effort must be maintained at all stages for obtaining useful data pertaining to the objectives sought.

B. Masonry Dams

1. Displacements

(a) *Plumb Line*.—Out of the five dams selected for review, installation of plumb line is included only in Gandhisagar and Linganamakki Dams. In the former, installation of the full assembly was in hand when seen during a site visit, and at the latter, installation had not yet been taken up. No data is available so far.

(b) *Geodetic Observations*.—Original instrumentation plans for all the projects under review did not include any geodetic measurements.

(c) *Joint Meters*.—Joint meters have been provided at Linganamakki Dam, Gandhisagar Dam, Nagarjunasagar Dam and Tungabhadra Dam out of the five dams under review. At Linganamakki Dam these have been provided across a transverse contraction joint in the non-overflow section, near the base of the dam. At Gandhisagar Dam they have been provided near the base at mid-height and near the top of the two transverse contraction joints in spillway section and one in power intake blocks.

In Nagarjunasagar Dam they are installed in the spillway block and one each of abutment blocks on either side of spillway. At Tungabhadra Dam these have been installed in a spillway block and in one non-overflow block. Jointmeters were installed to provide information on joint movement.

Lead terminations are in similar unsatisfactory condition as described under para B. 2 on page 9.

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Data have been collected at all the above dams except at Tungabhadra Dam where all the jointmeters have become inoperative. Data have not been processed except in the case of Nagarjunasagar Dam.

(d) *Foundation Deformation Measuring Devices.*—Original instrumentation for all the five dams did not include any means for observing settlement of the dam foundation.

(e) *Tilt Measurement Devices.*—No provision for installation of tilt measurement devices at any of the five dams under review was made in the original plans of instrumentation. This measurement actually supplements the information from plumb lines and geodetic surveys so far as gravity dams are concerned.

2. Stresses

Carlson type stressmeters have been embedded in Gandhisagar Dam, Linganamakki Dam and Nagarjunasagar Dam. At Gandhisagar Dam these have been provided near the base of maximum spillway block.

At Nagarjunasagar Dam these have been placed at the base of the maximum spillway block and an abutment block in non-overflow section.

At all the above three dams leads are not terminated at terminal boxes, and are hanging freely from gallery faces. At Linganamakki the leads have been frequently under water. Dry conditions at terminations of leads could not be ensured at any of the above three dams.

Data has been collected at all the three dams, but it has not been processed for the reasons that either instrument particulars regarding calibration were not identified as in the case of Gandhisagar Dam, or the cable terminals were not properly terminated, as in the case of all the three dams.

In the case of Linganamakki Dam, further observations have been suspended on account of the frequently submerged conditions of the lead terminals.

Comments regarding the conditions and maintenance of the test sets and measurement of grounding resistance made under para 1.2A.2 are pertinent.

The limitations mentioned in para B.3 regarding properties of masonry are also relevant.

No useful information can thus be obtained from the data observed at all the three dams on account of the conditions mentioned above.

3. Strains

Strainmeters of Carlson type have been placed in Gandhisagar Dam, Hirakud Dam, Nagarjunasagar Dam, and Tungabhadra Dam, out of the five dams under review.

At Gandhisagar Dam, they have been embedded at the base of maximum spillway section. At Hirakud Dam they are embedded near the base of the two spillway blocks.

At Nagarjunasagar they have been placed at base of maximum spillway block and at the base of an abutment block, on either side of the spillway portion. At Tungabhadra, strainmeters have been embedded near the base of the maximum section abutment block.

Lead terminations are in similar unsatisfactory condition as mentioned under para B.2.

Whereas data has been observed at all the above dams (being observed twice daily at Tungabhadra Dam) processing has not been started at any of these dams on account of the fact that (i) calibration data cannot be related to the various meters for want of identification, (ii) information on elastic and non-elastic properties of masonry are not available, (iii) initial readings not available, (iv) lack of personnel trained in processing of the observed data.

Besides these factors, the Committee considers that strainmeter observations in masonry dams are subject to the following basic limitations:—

- (a) Heterogeneity resulting from the size of rubble;
- (b) Uncertainty introduced by embedding the strainmeter in a small concrete block within the masonry mass the two materials having widely different elastic and non-elastic properties;
- (c) Insufficient information regarding elastic, plastic and creep characteristics of masonry.

(Refer to Carlson's letter dated March 10, 1965 Appendix I-1.02).

There is also lack of knowledge regarding shrinkage growth and thermal properties of masonry. Strainmeter observations on masonry dams, therefore, may not be useful, until above limitations are overcome by further research.

4. Pore Pressures

Carlson type pore pressure cells have been embedded at Gandhisagar Dam and Nagarjunasagar Dam. At Gandhisagar Dam they have been installed at mid-height in two spillway blocks. At Nagarjunasagar Dam these have been embedded near the base and mid-height in maximum spillway block. Lead terminations are in unsatisfactory conditions as stated under para B.2. Data has been observed at the above two dams but processing has not yet been done. Conclusions regarding the usefulness of these cells cannot, therefore, be drawn at this stage.

5. Temperature

Carlson type thermometers have been embedded at all the five dams under review, and are located at various points to provide information regarding the development and distribution of temperature within the body of the dam, at the upstream and downstream faces. At Tungabhadra Dam all thermometers have become inoperative and no data is available.

Data has been observed at remaining four dams under review. However, isotherms have yet to be drawn and temperature effects on structural behaviour have yet to be established.

6. Uplift Pressure

Of the five dams under review, uplift pressure pipes have been provided at Gandhisagar Dam and Hirakud Dam. Pipes have been provided in the spillway block and one power intake block. As there is no transverse gallery in each of the above two blocks, all uplift pipes are led to the observation gallery by bends and horizontal extension lengths. The pipes are capped and have not been provided with individual stop cocks and pressure gauges for record of uplift.

Monthly observations are made by connecting the pressure gauges directly to the individual pipes when the pipes start flowing. This method of measurement impairs the accuracy and reliability of uplift pressure measurement and realistic picture of uplift is not obtained. Uplift pressures observed so far, have been below the designed criterion values. As all the pipes have bends, redrilling of the uplift holes cannot be done even when required for ensuring normal functioning of the pipes. Nor can sounding be taken in case of uplift level below bends of the pipes.

7. Seepage Measurements

No measurement of seepage have been made available at any of the five dams under review :

8. Recommendations

From the general examination of data, results and interpretation of instrumentation for masonry dams, the Committee recommends as under :—

- (a) Sub-paras (a) and (b) of para 1.2-A.8 are applicable in toto.
- (b) Sub-paras (c) of para 1.2-A.8 is applicable with the following exceptions ?
 - (i) Under 'Displacement', exclude 'those determined by embedded resistance jointmeters at contraction joints where grouting is required to be done', since grouting of contraction joints in masonry dam is not required.
- (c) Sub-para (d) of para 1.2-A.8. specifying non-obligatory measurements, is applicable except that :

direct measurement of stress is not recommended for reasons explained under para 1.2-B.3 and para 3.2.2.

C. Earth Dams

1. Displacement

(a) *Surface Markers*.—This review is limited to observations on Hirakud Dam, Kotah Barrage and Gangapur Dam.

(i) At the Hirakud Dam, the surface markers were installed only on the parapet wall. No markers were provided on the berms on the downstream slope although planned (see Spec. & Schedule for Hirakud Dam).

(ii) At the Kotah Barrage and Gangapur Dam, surface markers on downstream slope were not planned. Settlement of parapet has been observed.

On Indian earth dams installation of surface markers has generally been found to be impracticable. This is because the dams are often built with truncated sections so that the section is widened as the embankment is raised. The upstream filling often follows soon after construction and in the interval between construction of upstream slope and filling of the reservoir placement of rip-rap is generally in progress. Consequently measurement of surface displacement during construction is generally impracticable.

Measurements of vertical settlements and horizontal movements parallel and perpendicular to axis constitute valuable data and the absence of such measurement is a serious drawback of the system of measurement used for the dams reviewed. In the absence of such observations no comments can be made regarding the tendency or otherwise for the slopes to bulge and the magnitude of longitudinal strains caused by settlement.

The measurement of settlements have generally been done by levelling to benchmarks on the abutments or on walls resting on rock. Although this introduces an element of error because of the regional settlements caused by reservoir load, interpretation of the behaviour of earth dams is not generally vitiated. The relative movements between embankment and the rocky substrata of low compressibility and relative movement between walls of spillway structures and embankment are generally sufficient for appraisal of the performance of earth dams. The surveying methods adopted for measurement of settlements and the dams reviewed are, therefore, considered adequate.

(b) *Internal Vertical Displacement (Cross Arms)*.—Cross arm data for Hirakud Dam, Kotah Barrage and Gangapur Dam was reviewed. The review of data from Jirgo, Nanaksagar and Danta Dam is based on the report published at the Eighth ICOLD (International Congress on Large Dams) 1964, by Gupta & Sharma.

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At Hirakud Dam a total of three cross arms were installed at two cross sections [(one at RD 9500(E) and two at RD 7250 (C&D)]. Two of these are still operative viz. D&F. Observations have been made weekly since installation (i.e. 1951 to 1956, construction period and thereafter). Cross-arm E in the deepest part of the dam provided valuable indications of the performance of the dam. The observations indicated significant consolidation of certain soft zones in the embankment during construction and thus served to resolve the uncertainty regarding the strength of such zones.

The foundation settlement at Ch. 9500 was 1.7% (1.66 ft.) of the embankment height in a period of 8 years. Of this only 0.8% (0.73 ft) occurred during the construction period of 2½ years and the balance 0.9% occurred after construction in 5½ years. The settlement of the embankment proper at this section (Ch. 9500) was 3.2% in 8 years of which post construction settlement was 2.4% in 5 years. At cross arms C & D where the foundation is incompressible for all practical purposes, the total settlements are 6.4% (11.3/178.2) and 5.9% (10.0/169.7) respectively. Out of this the post construction settlements are 1.6% and 1.4% respectively for a period of 5 years. The post-construction settlement is somewhat higher than the range of values experienced for USBR dams. For 11 dams investigated by Gould (Tech. Memo 648) the post-construction settlement is less than 0.2% for 3 years and is equal to 0.4% for 14 years (data only for denser broadly graded soils). In the absence of consolidation test data on embankment soils as placed nothing can be said about the relative magnitude of actual and anticipated settlement. The cross arm data, however, indicates that the post-construction settlements are well within the range of magnitudes for which the necessary freeboard can be conveniently maintained during maintenance.

Cross arm data have also indicated that the settlement is generally greater on the upstream side. Settlement during the filling of the reservoir is greater than the same in any other equal period.

At the Kotah Barrage two cross arms were installed at cross sections 410 ft. from the retaining wall, junction of the spillway and earth dam. The total settlement in the construction period of April to June 59 is 1.5% of the height (1.6 to 1.7). The post-construction

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settlement is reported to be of the order of 2%.

At the Gangapur Dam, 3 cross arms installations were made. Observations were made during construction 1950 to 1955 and they had to be discontinued as the torpedo get stuck up in one of the cross arms. This appears to be due to lateral displacements in the earth mass.

The procedure and equipment (i.e. torpedo) adopted for observations at Hirakud Dam has been found to be generally satisfactory. The equipment on all the projects is according to USBR design and is found to be convenient for field installation.

2. Measurements of Vertical Strains

(a) No attempt has been made at any of the three dams viz. Gangapur, Hirakud, Kotah Barrage to process the cross arm data to compute vertical strain and to study the compressibility of the fill material. Absence of consolidation test data on embankment soils precluded comparison of observed and anticipated compression. The system of soil classification adopted at Hirakud, Gangapur and Kotah Barrage makes it difficult to assess the degree of plasticity of the fines. The original data (especially the index properties) will have to be scrutinised afresh to reclassify the soils by the unified classification system. (The unified system was not a widely accepted standard practice when these dams were built.) In the absence of such classification it may be difficult to collect representative samples for consolidation tests and analysis even if fresh drilling or sampling is undertaken as a research programme.

At the Nanaksagar Dam (U.P.) laboratory compression data (from one dimensional consolidation test) was used to compute the relation between % compression versus pore pressure. This relation was compared with the observed relation between compression as measured on the cross arm installation and observed pore pressures (from piezometric data). This is a solitary instance among all the installations reviewed where comparison of pore pressure and cross arm data has been made to check their consistency. It would have been helpful to compare the laboratory test curves for effective vertical stress versus % compression (from the oedometer) and similar curves of observed compression versus computed vertical effective stress based on assumed vertical total stress distribution in the earth mass and observed pore pressure. Limited data reviewed by Gupta and Sharma indicates that observed pore pressures are less than those computed by Hilf's method apparently due to dissipation of pore pressure during construction. If allowance is made for dissipation of pressure of 1/7, the observed pore pressure would agree closely with pore pressures computed by Bishop's modification of Hilf's methods.

(b) *Horizontal Strain (Internal Displacements).*— (Horizontal cross arm or slope indicators): No provisions were made in any of the dams reviewed to measure the internal horizontal strain or displacement. It has been indirectly concluded from the verticality of cross arms at Hirakud that the horizontal displacement is insignificant after lapse of several years and in spite of considerable vertical settlement having taken place. The obstructions to the movement of the torpedo in Gangapur cross arms, on the other hand, seems to suggest appreciable horizontal displacement.

3. Pore Pressure Measurements

Measurements of pore pressures have been carried out either by simple stand pipe or by twin tube hydraulic piezometers. Porous tube (casagrande) type piezometers have not yet been used on any dams. Electrical piezometers have also not been used. Vibrating wire pore pressure cells have been procured for Ukai Dam but they have not yet been installed.

(a) *Stand Pipes*.—These have been used for appraisal of the performance of the grouted curtain in the pervious alluvium at Kotah Barrage and Girna Dam. Some stand pipes are also installed at Hirakud for verification of the phreatic line but very few observations have been carried out on these so far.

Recently stand pipes were installed at Gangapur Dam for observation of the phreatic line. The observations are not yet available.

Nine stand pipes were installed at the Kotah Barrage at 3 cross-sections, having one piezometer on the upstream and two on the downstream of axis of dam at each cross-section. The observations of water level in the stand pipe indicated a drop across the curtain of about 90% of the total head on first filling of the reservoir. Subsequent observations are being made on all the stand pipes. Unfortunately, the lag time or equalisation time of the stand pipes is not checked as a normal procedure. Hence, it has not been possible to ascertain when the stand pipes got silted up. Observations made in 1964 monsoon indicate abnormally long lag times. From the installation data it appears that the depth of penetration of the stand pipe into the highly pervious bouldery substrata is insufficient. The above limitations of the observations and the installation made it difficult to draw definite conclusions regarding the long term performance of the grout curtain. At the Girna Dam thirty stand pipes are installed in five cross sections at the rate of six in each cross section. Because a minimum of two stand pipes are installed on the upstream and four on the downstream of the curtain, definite indications of the pressure drop through the curtain and the head lost in upstream blanket are available. Observations are made every fortnight and lag times are checked before every observation. Stand pipes showing abnormal lag time are flushed by air and water jetting or replaced by new pipes when flushing is not possible. Observations provided useful data for appraisal of curtain performance during and after grouting. The drop across the curtain ranges from 85–90% of the head between the ground water levels at the downstream toe of dam and the pore pressure elevation upstream of the curtain (*i.e.* of the net head across the curtain after deducting the head lost in the silt blanket which is 10–15% of the total head).

Experience of stand pipe installations indicates that they are useful for measurement of the pressures in pervious saturated soils. However, proper planning of

the installation and adequate depth of penetration into the pervious substratum is essential to ensure reliable indication of the performance of seepage control measures. In the absence of initial measurement of lag time and frequent check in the lag time, the observations are likely to be erroneous. Large diameter stand pipes are especially vulnerable to sedimentation of silt in the large water column.

(b) *Hydraulic Piezometers*.—At Hirakud and Gangapur dams USBR twin-tube hydraulic piezometers were installed. At Hirakud the tubing and gauges were imported and the tips and fittings were fabricated indigenously. At Gangapur only the tubing was reported to be imported. When these installations were made (1950-55) the high air entry value ceramics were not developed for use as porous stones. This has given rise to errors and difficulties in measurement where the pore water pressure is negative either at the tip or the gauge anywhere within the tubing. In such cases, the water column cannot be held as for example in tips downstream of the cutoff below the gauge well level.

At Hirakud Dam the tubes were not filled with water at the time of installation and difficulties have been reported during subsequent draining and flushing. Even at 120 ft. head air could not be entirely removed from tip at elevations above the terminal well. The reading of some of the tips on the fringe of the phreatic line are found to be erratic (some are abnormally high) whereas a number of tips at levels considerably below the phreatic line continue to give consistent readings. For example, at RD 7200 in the deep channel portion pressure observations on tip numbers 6, 15, 16, 7, 17, 18, 25, 26 are consistent within themselves and indicate a gradual development of a steady seepage condition which is not much influenced by upstream reservoir fluctuation. This seems to be reasonable in view of the upstream position of the cutoff trench and the drainage relief afforded by the pervious sandy soils in the old river channel. Tip No. 32 which is on the fringe of the phreatic line seems to indicate high values inconsistent with the general pattern of the seepage phenomenon. Piezometric observations have been verified by a few stand pipes and the general indications are that the phreatic line under full reservoir conditions is well below the seepage line assumed for design. This is a very significant observation regarding the performance of the dams as the semipervious zone in the dam has almost the same permeability as the impervious zone. This observation demonstrates the efficacy of the downstream base filter and rock toe. Due to the erratic readings obtained in the hydraulic piezometers especially those on the fringe of the phreatic line, it is desirable to supplement the piezometric observations by additional stand pipes or porous tube installations. The porous tube piezometer by virtue of its lower lag time will give more reliable indications where the pore pressures are liable to fluctuate.

No data were collected regarding construction pore pressures at the Hirakud as the gauges were not available. Due to the small range of reservoir fluctuation (30 ft. in a height of 150 feet) erratic readings in some of the upstream tips, and lack of definite information regarding the seepage pattern under full reservoir conditions, definite indications regarding draw-down pore pressures are not available. The pore pressures in the upstream semi-pervious zone are found to rise with the reservoir but with some lag and it eventually attains the full reservoir pressure. The drop of pore pressure in the upstream semi-pervious zone for falling reservoir is much slower than the rise.

To ensure reliability of observations frequent and systematic checking of the performance of the meters is essential. Although 80% of the tips in the Hirakud Dam can be considered to be functioning (subject to the errors in the air entry value porous stone) the procedure adopted by the Hirakud Research Station, for checking the tips could be further improved on the lines suggested in Part II.

Gangapur Dam.—Hydraulic piezometers were installed during construction on three cross sections. The installation was planned to represent the deepest section, the zone of highly plastic clay foundation and the sections where the downstream inclined filter is not provided. The observations were carried out by the project staff from 1954 to 1962 and this work was taken over by the Maharashtra Engineering Research Institute in 1962.

A number of tips in the vicinity of the sloping filter and on the downstream have been giving erratic readings. Some of the readings are high especially in tips below the elevation of the gauge well. In this dam (as in Hirakud Dam also) the difficulties in getting consistent readings seem to be associated with negative pressures at the tips or anywhere in the measuring system, *i.e.* tube and gauges. High air entry ceramics may help to obviate this difficulty and facilitate filling of the system with water during installations. Definite indications regarding the phreatic line cannot therefore be obtained from the Gangapur data unless additional stand pipe or porous tube piezometers are installed. If the erratic readings in the vicinity of the filter and in the downstream are ignored, pore pressures in the hearing seem to agree with a flow net converging to the sloping filter. Gangapur piezometers can yield valuable data regarding draw-down pore pressures as the range of draw-down is about 50% of the height. These observations will be of great value especially because of the presence of soils of low permeability and high plasticity soils in the embankment and foundations. It is understood that such a study is planned by the Maharashtra Engineering Research Institute, in 1965-66 after thorough checking of the piezometers.

Out of the 80 tips installed only 29 tips can be considered to be indicating normal behaviour (including

tips subject to the errors due to air entry in the system) 46 tips are leaking and 15 are clogged. However, as the installation was well planned and large number of tips were provided useful data can still be obtained from about 20 tips regarding the draw-down conditions. If the piezometric installation is supplemented by fresh stand pipes and porous tubes, indications regarding the flow net and the phreatic line can also be obtained.

At the Nanaksagar Dam (U.P.), twenty five piezometers were installed on one cross section in 1962. The tips have provided good indication of the construction pore pressures which are reported to be below the values computed by Hilf's method* using compressibility data obtained from laboratory one-dimensional consolidation test. Fairly good agreement has been reported between the observed values and pore pressures computed by Bishop's step method** taking dissipation factor as 1/7. Dissipation of pore pressure during shut down appears to be small. This is the only case where serious effort is in evidence to collect the necessary consolidation data and compare the measured values with anticipated values of pore pressure. The observed pore pressures in feet of water are 1.1 H and 0.25 H respectively in the hearing and casing materials where H=height of overburden in feet.

Polyethylene tubing was used at Nanaksagar. This seems to have functioned satisfactorily apparently because of the special care adopted in protecting the tubing during installation. USBR type tips with plastic body and alundum stone of low air entry value were used. The observations in the hearing zone are considered to be reasonable and reliable due to high degree of saturation. The observations in the sandy casing involves an element of error due to presence of air which cannot be ascertained.

4. Earth Pressure Cells

No instruments have been installed so far on any of the dams for measurement of total stresses in the earth mass. Indirect evidence of the vertical stresses can, however, be obtained from strains measured in vertical cross arms provided that it is assured that there is no lateral yield. Unfortunately no measurements of horizontal strain are available to definitely establish the state of earth pressure *e.g.* at rest (no lateral yield) or action (no limit on lateral strain).

* Hilf J. W. Estimating Construction Pore Pressures in Rolled Earth Dams: Int. Conf. SMFE, 1948, Rotterdam, Vol. 3 p. 234-240.

** Bishop A. W. Some factors controlling the pore pressures set up during construction of earth dams: 4th Int. Conf. SM&FE, 1957, London Vol. II p. 294-300.

5. Recommendations

(a) *Obligatory Measurements.*—It is considered that the following measurements are obligatory for all earth dams over 50 feet high except where project authorities relax the provisions depending on size and importance of the dam:

- (i) Internal vertical settlement and strain during and after construction.
- (ii) Surface settlement and horizontal displacement after construction. Wherever possible these observations may be done during construction also.
- (iii) Stand-pipes and porous tubes piezometers for appraisal of seepage control measure both in the body of the dam as well as through the foundations. These must be combined with the measurement of seepage.
- (iv) Hydraulic piezometers in regions liable to development of high positive pore pressures during construction and in the upstream casing and semi-pervious zones of dams with large range of draw-down.

The above instrumentation is considered necessary as a measure of safety as apart from observations made for advancement of knowledge regarding strength and stability of earth masses as well as the seepage phenomena through embankments and their foundations.

Settlement: External & Internal.—Settlement data during construction is essential as it provides indications of the rate of consolidation and any tendency for cracking or differential settlement. As surface settlement observations have generally been found to be impracticable during construction, cross-arms are the only means available for observation of settlement during construction and filling of the reservoir.

When adequate data for identification of soils and representative consolidations tests results are available, vertical compressive strains computed from cross-arm observations permit checking of the pore pressures by comparison of the field compression characteristics (effective stress upstream strain) with the laboratory consolidation test results (e-p curves). Cross arms observation also provides an overall appraisal of the quality of earth work as abnormally slow settlement or excessive settlement indicates wet placement or low density respectively.

Post construction observation of horizontal surface movement is essential to ascertain whether there is any tendency for the embankment to bulge. Potential tendency for cracking transverse to axis can also be verified by observation of longitudinal surface strain.

Horizontal cross arm installations are necessary when appreciable horizontal movements are anticipated due to plasticity of the foundations and embankment soils.

Wherever the stresses in the earth mass are to be investigated, installation of vertical as well as horizontal cross arm is essential. Measurement of horizontal strains is also essential in the vicinity of retaining walls to investigate the state of stress (actual or at rest).

Pore Pressure Measurements.—Stand-pipes and porous tubes can be installed at any stage. As the procedure for checking of stand-pipes and porous tubes is relatively simple, they should form an integral part of the system of observations of seepage phenomena.

Tubing for Piezometers.—For observation of draw-down and construction pore pressure hydraulic twin tube piezometers are essential as no other instrument is capable of providing the necessary data with equal reliability. At the present stage tubing of proved quality is not available indigenously. While some of the draw-backs of the indigenous material (PVC) have been overcome by careful installation (avoiding small radius bends) in some of the recent dams (e.g. Bor), the length of experience on this type of tubing is not adequate to confirm its satisfactory performance beyond 5 years. It is likely that PVC tubing may perform better in India as compared to cooler climate abroad. Until tubing of proved quality (PVNC) or nylon polythene is manufactured locally, available material (PVC) may have to be used which may deteriorate in 5-10 years. In a majority of earth dams this may serve the purpose as the pore pressure observations after initial period of 5—10 years may be required only regarding the downstream foundations seepage phenomena in the foundation or the embankment (i.e. the phreatic line). Porous tubes installed after construction may serve the purpose in such cases.

Porous Stones.—The necessity of high air entry value ceramics may also be obviated by use of porous tube piezometers in the zones of negative pore pressure in the embankment and downstream foundations. Ceramic stones of high air entry value manufactured locally have also not been tested sufficiently. When such stones are used there is risk of some of the stones being impervious. Careful and thorough testing is, therefore, essential if indigenously manufactured ceramic stones are to be used. In zones of high construction pore pressure, porous stones of low air entry value may be used as satisfactory performance of early USBR piezometers (with carborandum porous stone) has confirmed their utility in zones of high positive pressure. Wherever piezometers of low air entry value are used, it is essential to verify the reliability of pore pressure observations by comparing the field compression curves (computed total pressure minus observed pore pressure versus arm strain) with the laboratory compression curves (e-p) corrected for gravel contents.

In important dams where long term satisfactory performance of piezometers (in the upstream portion of the embankment) is obligatory, tubing of only proved quality should be used alongwith ceramic of high air entry value.

TERM OF REFERENCE-I

(b) *Special Measurements.*—The following additional instrumentation would be necessary in earth dams according to the special conditions obtaining at particular sites :—

- (a) Measurements of horizontal strain would be necessary where significant movement is expected due to plastic nature of the soil in the embankment or foundation. Measurement of horizontal strain should also be necessary where progressive failure is apprehended due to significant difference in the stress-strain characteristics of soils in different zones or different parts of the foundations.
- (b) When the design assumptions are tentative due to complexity of the phenomena, installation of inclinometers (slope indicators) would be desirable to ascertain presence or otherwise of tendency for dangerous internal horizontal movement. Similar installation would obviously be essential in test embankment as inclinometer (slope indicator) can clearly indicate the location of the slip surface.
- (c) When problems of long term stability are involved, an element of uncertainty always remains in the estimation of shear strength. All available means of detecting movements such as inclinometer (slope indicator) horizontal cross arm and surface markers must be used. Continuous review of the observation is also essential to reveal departure, if any, from the anticipated behaviour so that corrective measures can be taken in good time before dangerous conditions develop.
- (d) Inclinometer installation can provide very useful indication of tendency for separation between core and the abutment. Hence it would be obligatory for :
 - (i) abutments with steep and irregular profile;
 - (ii) high dams with narrow cores.

As instruments for measuring total earth pressure have still not been proved regarding their reliability, very great care in installation, observation and interpretation of results of such instruments is necessary. Observations of total earth pressure by means of electrical resistance stress meters or vibrating wire stress-meters must, therefore, be considered as part of research programme.

D. Other Structures

Apart from dams of various types, river valley projects involve several other structures for which instrumentation may be required for safety and future economies in design. These may be classified into two broad categories :

- (a) Tunnels and rock structures ;

- (b) Reinforced concrete and prestressed concrete structures.

The first category includes tunnels for various purposes such as pressure tunnels, access tunnels, diversion tunnels, dam foundations and cavities in rock for underground powerhouses and surge shafts. The second category includes aqueducts and bridges, powerhouse structure, openings such as shafts sluiceways and galleries in dams, conduits, spillway piers, towers and other framed structures. Each of these structures although small in comparison with dam, constitute a large part of the total cost of the project.

The techniques used and the nature of measurements is widely different in this group and choice of measurements depends on the variety of factors enumerated in Part II.

1. Displacements

(a) *Mechanical Methods.*—Measurements of displacements of this group of structures has been made in almost all cases by dial gauges. In the case of generator floor of Koyna Power House, deflection measurement of the slab with and without the shaft load on the temporary support are proposed by means of sensitive dial gauges. In the case of Koyna Pressure Shaft deflections are not measured directly but may be computed from strains in the lining. Quite elaborate deflection measurements however have been made for the Kunnu Syphon aqueduct. Eight dial gauges were used for measuring deflection of the pipes at three locations, expansion of the span, settlement at bearings and the change in horizontal diameter of the pipes. It was found that the instantaneous deflections agreed quite well with the calculations. The centre of span deflected by 0.125 inch when the pipe was full compared with 0.1 inch predicted by the model. The pipe on ground settled by 0.15 inch at full load and increased in diameter by 0.18 inch as compared to the calculated value of 0.175 inch to 0.18 inch respectively. The tests also indicated that the neoprene bearings offered complete freedom of the horizontal movement of the pipe. In the Nagarjunasagar Right Canal Tunnel, deflection measurements have not been made as such, although the data would have been very useful.

(b) *Optical Methods.*—Optical methods have not been used in any of the structures mentioned except for the Kunnu Syphon aqueduct. Precise levelling was carried out from two fixed levelling instruments and the deflections were measured at eleven points over the pipe. These have indicated deflections of large magnitude.

(c) *Angular Changes.*—Angular changes have been measured in the Kunnu Syphon aqueduct at the collars by precision (1/10,000) levelling instrument which was permanently installed on the top of the collar. The test results showed that with 4 ft. of water in the pipe the slope changed 4×10^{-4} .

In none of the other structures the angular changes were measured.

2. Stresses

Direct measurement of stresses has not been attempted in any of these structures. In the case of Koyna Power House machine floor direct measurement by means of Carlson type of stressmeters would be expected to give valuable data but difficulty of procurement of the equipment has come in the way of such installations.

3. Strains

The greatest attention in all these structures has been paid to the measurement of strains. The stresses are computed from strain after taking into account the elastic property of the materials. The strains have been measured mainly by the mechanical and electrical type of gauges.

(a) *Mechanical Measurement of Strain.*—Mechanical strain gauge has been used only at Nagarjunasagar Right Canal Tunnel. The instrumentation in this tunnel has been planned to determine the rock pressures transferred to the steel supports. Studs were fixed on the supports at various places and the changes in strain with time were measured. In these tests, it was reported that the increase in load on the supports, after the time the tests were commenced, was negligible. However, in tests of this type more extensive data should be available before definite conclusions could be drawn.

(b) *Electrical Measurement of Strain.*—In all these four structures electrical measurement of strain has been or is to be carried out.

For the installation of electrical resistance gauges (Concrete gauges) at the generator floor of the Koyna Power House, extensive model tests have been carried out to locate the best position and alignment of the gauges which are to be embedded in the concrete. The tests are to be conducted both under static and dynamic loads with a view to determine the validity of the various assumptions in design analysis for this complex structure and the natural frequency of the structure.

In the instrumentation of the Koyna Pressure Shaft extensive use has been made of the unbonded type of electrical resistance gauges. These gauges are put at several locations in the concrete lining and also to the steel lining by attaching the ends to welded fixtures. Observations have been taken during the filling and emptying of the pressure penstocks.

The results are of considerable importance. The sharing of the load between the steel liner and the concrete has been estimated as 20:80. The original

design assumption was 50:50. From the strain data while unfilling, the buckling pressures on the lining has been estimated as 20% of the head. The rate of dissipation of this pressure has also been determined. These observations in the Koyna pressure tunnels may prove of great utility in improving future designs of linings.

In the case of Kunnu Syphon aqueduct bonded wire electrical strain gauges were used. Strains were measured at eleven locations by means of paper bonded gauges. Recording and measurements were conducted by means of ten-way static recorder and four channel dynamic strain recorder. Due to the incompleteness of temperature compensation, uncertainty about creep and shrinkage, uncertainty in binding, the quantitative interpretation of strain data has not been possible. However, certain qualitative interpolation regarding the start of cracking has been made.

In Nagarjunasagar Tunnel attempts had been made to measure the strains by means of electrical bonded wire strain gauges but these were later discarded in favour of mechanical gauges.

4. Dynamic Measurements

Dynamic measurements in structures other than dams involve the measurement of dynamic strains on the structure as well as the measurement of natural frequencies. Comparatively little work has been done in this field. One of interesting studies in this connection was the measurement of dynamic strain at the roof of Bhira old tunnel when blasting was going on at the adjacent new tunnel. The strain measurement indicated the order of the charges to be used in blasting. Similar work has been done in recent years in Hirabhasgar and other projects.

5. Temperature

Temperature measurements in these structures have not generally been conducted. However, strainmeter installed in Koyna pressure tunnels give the temperature as well.

Measurement of ambient temperature, surface temperature and the interior temperature may be quite important, for structures like intake towers exposed partly to hot sun and partly submerged in water. Thermal stresses may be very high under such condition. No systematic measurements seem to have been made so far.

6. Visual Observations

Visual observations of cracks if any and their development with time has been done in most structures but systematic records have to be kept.

TERM OF REFERENCE-I

7. Recommendations

After reviewing the instrumentation in structures other than dams, involved in river valley projects in the country, and comparing the work in this field conducted in other countries, the Committee makes the following recommendations :

1. Although instrumentation for concrete and rock structures need not be obligatory, more extensive instrumentation in selected structures will pay great dividends. The instrumentation in Koyna pressure penstocks have already indicated how less conservative assumption could be used in the future designs, thus effecting considerable savings in material and cost.
2. For this type of structures no standard instruments or measurement procedures need be prescribed. Instrumentation for each case has to be tailor-made. Deflections could be measured by optical, mechanical or electrical gauges. Strains and stresses could be measured by mechanical or electrical method. For long term studies unbonded electrical resistance gauges and vibrating wire gauges prove suitable but for short term observations bonded SR-4 type gauges could be extensively used. Measurement of accelerations, natural

frequencies of the structure and dynamic displacement and strains are also of value in improving future designs.

3. Association of the structural designs and research workers in the initial stages of planning the instrumentation is absolutely necessary. The objectives of the instrumentation programme must be clearly formulated in relation to the existing methods of analysis and design. The various parameters including the loads and properties of materials must be visualized and instrumentation designed accordingly.
4. Prior to the installation of instruments in these structures, analytical and model studies should be carried out. Model studies are particularly valuable for indicating the order of magnitudes of the quantities to be measured and the optimum locations of the instruments. For example model studies of Kunnu Syphon aqueduct and of Koyna Power House Structure have proved of great assistance in planning the installation and in interpreting the results.
5. Wherever major structures are being demolished, instrumentation in the structure to determine the behaviour of the structure up to failure is desirable.

2.0. TERM OF REFERENCE—II

“To assess future requirements of instruments upto the end of the Fourth Plan with particular reference to the requests of the projects now under construction.”

The total assessment of the requirements has been worked out for the projects spilling over from the Third Plan period on the basis of available data. Sufficient data, however, is not available for the new projects to be taken up during the Fourth Plan period. An *ad hoc* provision has, therefore, been made for the projects as well as for research and training programme requirements. Requirements of instruments for special measurements in structures other than dams and laboratory research are not estimated. The number of such instruments will be small.

In keeping with the recommendations for the term of reference (i), the assessment of future requirements is based on the following obligatory measurements :—

(a) Concrete and Masonry Dams

- (i) Deflection
- (ii) Joint Opening
- (iii) Uplift Pressure
- (iv) Temperature

(b) Earth Dams

- (i) Surface displacements and settlement
- (ii) Vertical internal movements, including foundation settlement
- (iii) Pore Pressures

For a few projects, as detailed in Appendix I-2.01, the following additional measurements have also been provided for :—

(a) Concrete Dams

- (i) Stress
- (ii) Strain

(b) Masonry Dams

- (i) Strain

(c) Earth Dams

- (i) Internal horizontal movement and strains

(d) Other Structures

- (i) Deflection
- (ii) Stress
- (iii) Strain
- (iv) Temperature
- (v) Contact earth pressure

The following is the abstract of the total requirements tabulated at Appendix I-2.01 :—

(A) Concrete and Masonry Dams

(i) Wilde T3 Theodolite	15
Wilde N3 Level	15
(ii) Deflection Measuring Equipment	
(a) Plumb Lines	70
(b) Micrometer Slide }	35
Microscope	
(iii) Jointmeters	800
(iv) Stressmeters	45
(v) Strainmeters	1200
(vi) Pore Pressure Cells	40
(vii) Bourdon Pressure Gauges	1000
(viii) Resistance Thermometers	1000
(ix) Instrument Reading Sets	35

(B) Earth Dams

(i) Surface settlement points	1830
(ii) Cross arms	
(a) Vertical	3500
(b) Horizontal	125
(iii) (a) Inclinator installation	40
(b) Slope indicator probe	710
(iv) Base plates	750
(v) Piezometers	
Foundation type	2,000
Embankment type	3,800
Porous tube	670
(vi) Earth pressure cells	
(a) Contact pressure cells	30
(b) Total pressure cells	30

(C) Other Structures

(i) Deflection measuring equipment	
Theodolite & Wilde T3	10
Wilde N3 level	10
Dial gauges	500
Invar Tape	12
Settlement gauges	25
Inclinometers	8
(ii) Stressmeters	20
(iii) Strainmeters	
SR-4 gauge	50,000
Carlson strainmeters	25
Vibrating wire strainmeters	25
Huggenburger strain gauges	15
(iv) Resistance thermometers	100
(v) Test sets	8

3.0. TERM OF REFERENCE—III

“To examine different types (such as Carlson elastic wire, vibrating type etc.) of instruments and recommend the best suited for our projects.”

The term of reference is related to—

- (a) conclusions drawn from the preceding general review and examination of data, and
- (b) the suggestions of the Committee with regard to progressive manufacture of these instruments indigenously [term of reference (vii)].

The question regarding types of instruments best suited for our projects has, therefore, been considered in the light of measurements recommended as being obligatory for all dams or optional under special circumstances. For measurements which are obligatory, the number of instruments in each category would be much greater than for the optional installations. As such, their availability within the country now or in the future would be of greater significance than for the relatively smaller quantity of those instruments warranted only under special circumstances.

A description of instruments generally used for various measurements of structural action is given in Chapter 3 of Part II of this report. This forms the basis for the following discussion and recommendations.

3.1. INSTRUMENTS FOR MEASUREMENT OF DISPLACEMENT

3.1.1. Concrete and Masonry Dams

3.1.1.1 For Geodetic Observations

Geodetic observations of displacement have been recommended as being obligatory for all concrete and masonry dams. Such observations are required to be made with respect to a distant point of reference, reasonably unaffected by the dam and reservoir loads. A high degree of accuracy is called for, particularly as the displacements would be of a small order. The instruments recommended for this purpose are the equivalent of the Wilde T-3 Theodolite and Wilde N-3 Levels, with the matched accessories supplied by the manufacturers. Specifications for the Wilde instruments are at Appendix I-3.01 and I-3.02 respectively.

3.1.1.2 For Plumb Line Observations

Observations of Plumb Lines (including the reverse plumb line) can be made by either—

- (a) Micrometer slide and Microscope as used at Bhakra Dam

- (b) Huggenburger Coordimeter as used at Koyna Dam
- (c) Huggenburger Kleidoscope as used at Rihand.

All three instruments can be expected to yield reliable observations. For our projects the Committee recommends the Micrometer slide and Microscope which is the simpler and more rugged of the three, and for which there are better prospects of indigenous manufacture with least imported components. Specifications for the Micrometer slide and Microscope obtained for Bhakra Dam are given at Appendix I-3.03.

3.1.1.3 For Joint Movement Observation

(a) *Surface Gauges.*—Where surface measurements of joint movements at accessible locations are required, mechanical gauges of the Whitemore-Huggenburger type are considered by the Committee to be reliable, and suitable for our requirements. The specifications for this gauge obtained for Koyna Dam are given at Appendix I-3.04.

(b) *Embedded Gauges.*—Observation of joint movement by means of embedded gauges can be made by either the Carlson elastic wire jointmeters or the vibrating wire types of instruments. The Committee is of the opinion that both these types of instruments for observations of joint movements are comparable in respect of operating and sensitivity characteristics and can be expected to yield equally reliable results. In respect of progressive manufacture of the instruments and test sets within the country and dependence on imported components, there would not be any significant advantage of one type over the other.

Jointmeter installations in India have so far been confined to the Carlson elastic wire type of instruments by virtue of which a certain degree of familiarity has developed in respect of installation procedures, observations and processing of data as also in respect of the test sets used. Considering the existing conditions in the country in matters relating to instrumentation, the committee recommends that the continued use of the Carlson elastic wire type of jointmeters would be desirable, subject to limitations of moisture conditions mentioned in para 3.3.1. Specifications for the Carlson elastic wire type jointmeter are given at Appendix I-3.05.

3.1.1.4 For Observation of Foundation Deformations

For this purpose long gauge instruments called foundation deformer are required. For these instrument basic measuring element is a joint meter or strainmeter depending upon the range and sensitivity requirements.

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Jointmeters or strainmeters are also used for observations of foundation deformation. As such the recommendation of the Committee regarding type of instrument best suited for our projects is the same as that given in para 3.1.1.3 for jointmeters and in para 3.3.1. for strainmeters.

3.1.1.5 For Observation of Tilt

Where observation of tilt is warranted by the special circumstances of a project, such as for arch dams, or on account of foundation conditions, tiltmeters or clinometers may be used. For measurement of tilt at the surface or in shafts and galleries, optical instruments would be appropriate. For measurement of tilt at interior points of the dam, electrical instruments of the vibrating wire type are suitable. Sample specifications of these instruments are given at Appendix I-3.06 and I-3.07. For higher sensitivity, seismic type of tiltmeters are to be preferred. Specifications of the seismic tiltmeter installed in Bhakra Dam are given at Appendix I-3.08.

3.1.2. Earth Dams

3.1.2.1 For Geodetic Observations

In view of larger displacements in earth dams, normally used surveying instruments should prove satisfactory for geodetic observations. However, for important structures and where the bed rock is deformable, greater precision in observations is required for which the instruments listed in para 3.1.1.1 would be necessary.

3.1.2.2-For Internal Vertical Movement Observations

For internal vertical movement, the conventional cross-arm of USBR design fabricated indigenously, has generally proved practicable and satisfactory. For specifications Designation E-29 of USBR Earth Manual (1963) may be referred. While the Committee recommends this type of installation, it would be desirable to develop for indigenous manufacture the more recent designs such as the induction or electromagnetic probe for ease of installation and observation in view of the difficulties experienced in operating torpedos at some installations.

3.1.2.3 For Internal Horizontal Movement Observations

Internal horizontal movement in embankment and foundations may be observed either by the USBR type of horizontal cross-arm or by inclinometers, or by the "Floating Concrete Rings" (Swedish Well). The horizontal cross-arm being basically of the same construction as the vertical cross-arm, should not present any difficulties either in indigenous manufacture or installation and the Committee recommends its adoption. For specifications of horizontal cross-arms, Designation E-30 of USBR Earth Manual (1963) may be referred.

The "Floating Concrete Rings" do not involve any special equipment. This installation can be located only downstream of the impervious core for observing horizontal movement of the embankment. For details reference may be made to page 399 of "Earth and Earth-Rock Dams" by Sharard.

The inclinometer installation consists of embedded extruded aluminium piping, a probe and measuring set. The piping can be manufactured indigenously but the latter have to be imported for the present. This installation is recommended where critical locations of significant relative movement need to be identified, such as steep and irregular abutment contacts and weak zones in the foundation. Specifications for Wilson Slope Indicator are given at Appendix I-3.09.

3.1.3. Other Structures

3.1.3.1 Optical Measurements

The most convenient instrument for measurements of displacement in structures other than dams for river valley projects are optical levels, theodolites or cathetometers. The choice depends on the respective magnitudes of deflection and the distance of the observation target to the point of observation. In case of bridges and aqueducts taking the above factors into account the Committee recommends the use of instruments similar to Wilde T-2 or T-3 theodolite.

3.1.3.2 Mechanical Measurements

The most widely used instruments for mechanical measurements of displacement in other structures are—

- (a) Invar tape or rods
- (b) Dial gauges
- (c) Microkaters
- (d) Whittemore—Huggenburger strain gauges
- (e) Liquid level indicators
- (f) Clinometers and precision levels.

For the measurements of the changes in diameter of tunnels due to rock movements, the Committee recommends the use of invar tape or rods with dial gauges. These have been successfully used in the rock stability studies at Niagara Tunnels. Dial gauges of various types and sensitivities ranging from .001" to .0001" are available and are most commonly used means of displacement measurements in other structures. Typical specifications for a dial gauge are given in Appendix I-3.10. While the dial gauges of the gear type are quite satisfactory in the medium sensitivity range (.001"), those of greater sensitivity tend to show erratic behaviour in higher sensitivity range under field conditions. The Committee recommends the use of Johnson Microkater type instruments for higher sensitivities. Typical specifications for Microkater type gauges are given in Appendix I-3.11. The Committee recommends the use of Whittemore Huggenburger type of

strain gauge for measurement of vertical and horizontal displacements across joints in aqueducts, bridges, spillway piers etc. For specifications of such a gauge refer to Appendix I-3.04. Where settlement of structures or footing of the order of a fraction of an inch are involved, the Committee recommends the use of liquid level indicators. Angular changes should be measured by means of special levels or clinometers. Special instruments will often have to be locally devised to suit a particular problem. For measurement of foundation disturbances caused by grouting Ontario Hydroelectric Power Commission has devised a special tiltmeter using two highly sensitive levels.

3.1.3.3 *Electrical Measurements*

For measurements of displacement at points which are inaccessible or are subject to hazards, the Committee recommends the use of electrical instruments where observations can be taken at a distance. In most of these instruments a bonded wire strain gauge is used as a transducer. Displacement pick-ups are available with the major strain gauge equipment suppliers. For short term test the Committee recommends that these gauges be locally constructed from SR-4 Straingauge elements.

3.1.3.4 *Dynamic Measurements*

In structures subject to impact or oscillatory loads, dynamic displacements should be measured by induction piezoelectric or other types of equipment. The most common type used at present is electromagnetic displacement pick-ups and these have proved satisfactory.

3.2. INSTRUMENTS FOR MEASUREMENT OF STRESS

3.2.1. For Concrete Dams

As the basic measuring element in instruments for direct measurement of stress is the same as in a strainmeter the recommendations of Committee are those given in para 3.3.1. Specifications for Carlson elastic wire type stressmeter are given in Appendix I-3.12, and those for vibrating wire type meter are given in Appendix I-3.13.

3.2.2. Masonry Dams

At the present stage of development, precise means for directly measuring stress in masonry dams are not available. The Committee does not recommend use of any of the known types of stressmeters in masonry dams, until possibility of direct stress measurement is established by a special research programme.

3.2.3. Earth Dams

Measurements of total earth pressures by instruments embedded in earth mass is considered only as a part of research programme. Hence no general recommendation can be made at the present stage of development.

3.2.4. Other Structures

For the reason stated in para 3.2.1 above, the recommendations of the Committee for direct measurement of stress with Carlson elastic wire and vibrating wire type of gauges are the same as given in paras 3.3.1 and 3.3.4. Direct measurement of surface stresses can be made by a modification of the bonded resistance wire gauge known as the "Dydic Wire Gauge". Specifications for this type of gauge are given in Appendix I-3.14.

3.3. INSTRUMENTS FOR MEASUREMENT OF STRAIN

3.3.1. Concrete Dams

Measurement of strain in the interior of concrete dams can be done by the Carlson elastic wire type or the vibrating wire type of instruments. As far as the embedded instruments are concerned, both types are comparable in ruggedness of construction, sensitivity, and long-term reliability. The following tabulation* lists some of the points for comparison of these two types of meters.

Item	Elastic wire meter	Vibrating wire meter
Meter leads.	Four conductor rubber-covered cable; relatively easy to embed; lead lengths limited to about 250 feet.	Two conductor copper shielded cable; rather difficult to embed because of lack of flexibility; lead lengths limited to about 1 mile.
Electrical shorts to ground.	High resistance to ground must be maintained to obtain accurate readings.	Accuracy of reading does not depend on maintaining high resistance to ground.
Test set.	Simple electrical circuit; rugged components, suited for field environmental conditions; can be repaired in field; easy to operate.	Electronic circuit; components affected by temperature, changes and shock; difficult to repair or adjust in field; moderately difficult to operate.
Meter temperature.	Data obtained directly by test set.	Data obtained directly by test set for short leads; vibrating wire thermometers required for long leads.

After careful examination, the Committee recommends that Carlson elastic wire type is to be preferred when sufficient precautions can be ensured against ingress of moisture into the cable. Although in the above table a limit of about 250 feet for the lead length is indicated longer leads up to 500 feet have been adopted at some installations without seriously impairing the accuracy of the measurement. At installations involv-

*Extracted from 'European Practice in Design and Construction of Concrete Dams' Vol. 3, by Copen Rouse and Wallace, U. S. B. R. March 1963.

ing longer leads and when there is a risk of moisture finding access to the cables, the vibrating wire type is more suited. Specifications for Carlson elastic wire strainmeters are given in Appendix I-3.15 and for the Maihak Vibrating Wire type in Appendix I-3.16.

3.3.2. Masonry Dams

In Masonry dams, at the present stage, measurement with standard gauges (10-20 cms) strain-meters of either type may not yield any significant results. Specially fabricated long gauge strainmeters which may use a strainmeter or a jointmeter as the measuring unit, depending on the range of movement expected, can however, be used for establishing whether any useful purpose would be served by such measurements. This is on account of the uncertainties mentioned in para 3.2.2. The choice of the measuring unit is to be guided by the recommendation in para 3.3.1.

3.3.3. Earth Dams

Vertical and horizontal cross-arm measurements can also be used for estimation of strain in embankments and foundations.

3.3.4. Other Structures

For surface strains in other structures the Committee recommends the use of SR-4 type bonded wire gauges, including concrete gauges. For easily accessible surfaces the mechanical Whittmore-Huggenburger extensometer would prove most convenient.

For internal strains the Carlson type resistance wire gauges are recommended where the leads are small and the vibrating wire gauges where leads are long as in tunnels. Specifications for SR-4 gauges are given in Appendix I-3.17. For concrete gauges specifications are given in Appendix I-3.18.

3.4. INSTRUMENTS FOR MEASUREMENT OF PORE PRESSURES IN THE INTERIOR AND UPLIFT PRESSURES AT FOUNDATION LEVEL

3.4.1. Concrete and Masonry Dams

The behaviour of pore pressures cells in concrete and masonry dams at installations has not yet been proved to be sufficiently reliable, and some doubt has arisen regarding their utility. Therefore, at this stage, wherever pore pressure cells have already been installed, a check on the results may be obtained by other means. Until such verification is available, use of pore pressure cells should be restricted.

For measurement of uplift pressure at the base of dam, uplift pipes should be provided at suitable locations. These should be vertical pipes terminating into a transverse gallery to facilitate maintaining these pipes in proper functioning condition by reaming and

redrilling when warranted. Bourdon type gauges should be permanently mounted on individual pipes terminating in the gallery.

3.4.2. Earth Dams

The Committee recommends the twin-tube hydraulic piezometer for normal measurement of pore pressure in earth dams and foundations, the high air entry value stone being necessary. However, it admits of being replaced by porous tube or stand-pipe installations in the downstream portion of the structure. Specifications for hydraulic piezometer are given in Appendix I-3.19. An important consideration is the quality of tubing used. The PVC tubing available indigenously has not yet been proved for long term use. Either import of the PVNC tubing or of PVNC compound is indicated until an equivalent indigenous substitute is found for ensuring that pore pressure measurements are not rendered doubtful on this account. Generally gauges supplied indigenously have not proved satisfactory, and pore pressure measurements are dependent on import of Bourdon type gauges for use as master gauges. Other accessories for the hydraulic piezometer can be manufactured in the country and further development in this is called for.

3.5. INSTRUMENTS FOR MEASUREMENT OF TEMPERATURE

3.5.1. Concrete and Masonry Dams

Out of various types of instruments available for measurement of temperature, the Committee recommends the Carlson resistance thermometers as being suitable for our projects. Specifications for the resistance thermometer are given in Appendix I-3.20.

3.5.2. Other Structures

In addition to the resistance wire type, measurement of temperature for structures other than dams can also be done by thermocouples which may be more suitable in certain cases on account of the space required for insertion in a structure. Specifications for thermocouples are given in Appendix I-3.21.

3.6. INSTRUMENTS FOR MEASUREMENT OF APPLIED LOADS

Measurement of dynamic load resulting from waves, earthquake and wind are excluded from this report. Only other load which requires special instrumentation is the earth load.

Earth pressures have not been measured so far in India in any of the dams. Hence no recommendations can be made regarding suitability of instruments for measurement of earth pressures on the basis of Indian experience. Instruments for this purpose should meet the basic requirement of covering as large area as

TERM OF REFERENCE-III

possible and recording the earth pressure with the minimum of movement which would be of the order of 1/2000th of the diameter of the measuring cell (for details see part II).

The measuring element in earth pressure cells consists of a strainmeter of the vibrating wire type or the unbonded elastic wire (Carlson) type. Hence recommendations of para 3.3.1 regarding choice of measuring instruments would apply to earth pressure cells also. It is generally difficult to restrict the length of leads

and hence vibrating wire instruments are better suited for measurement of earth-pressures.

Techniques for measurement of rock loads in underground structures are still in the process of development and Indian experience of such measurement is meagre. Hence it would be premature to make any specific recommendations in this respect.

The Committee would, therefore, stress the necessity for special efforts to develop required instrumentation for this purpose.

4.0. TERM OF REFERENCE—IV

“To recommend the best arrangement for recording, storage, analysis and interpretation of field readings from instruments”.

Having seen, at first hand, the arrangements at various projects in the country for recording, storage, analysis and interpretation of field readings from instruments, the Committee is led to the conclusion that a certain minimum degree of standardization must be ensured for this important and vital aspect of work. Wherever arrangements have been found wanting, the situation has arisen out of the fact that either personnel have not been set apart for this assignment or facilities have not been provided for their acquiring familiarity with and experience in structural behaviour observations and techniques. An active and concerted effort is called for on the part of the State Governments and project authorities if any worthwhile results are to be achieved.

The recently organized Instrumentation Cell in the Central Water & Power Commission, has contributed to some extent in bringing to the notice of various concerned agencies the value and importance of this work, besides rendering technical advice on the subject wherever requested by various project authorities. It is necessary to substantially augment this cell to enable effective coordination, collection and interpretation of vital information pertaining to the large number of projects completed or in hand.

The Committee has considered this matter and discussed it with concerned personnel in several States. As a result, the Committee recommends as follows :—

(a) That each State Government constitute an adequately staffed organization to perform the following functions :—

- (i) planning of the scheme for measurement of structural action on all projects where such measurements are warranted ;
- (ii) preparation of specifications of instruments and equipment to be procured ;
- (iii) preparation of detailed instructions for the installation of instruments and equipment and observations ;
- (iv) testing and checking of instruments procured ;
- (v) supervising the installation of instruments ;
- (vi) ensuring that initial readings are properly observed, and that necessary arrangements for subsequent continuous observations are made ;
- (vii) processing of data obtained ;

(viii) interpretation of results ; and

(ix) compilation of periodic structural behaviour reports.

(b) That the organization for structural behaviour studies constituted in each State should preferably be part of the Design Organization and maintain close liaison with research and quality control activities.

(c) That personnel for the organization to be set up in each State be adequately trained to perform the functions required. For this purpose, regular training and refresher courses should be organized. The Director, Central Water & Power Research Station, Poona has offered necessary facilities in this respect. These courses for officers of the rank of Assistant Engineer should include lectures, demonstrations, and practical work by personnel with requisite knowledge and experience drawn from various projects. Such courses should be held twice a year for a period of 6 weeks each for 15 officers at a time and should include instructions in installation of instruments, observation procedures, post-installation checking, and processing of observed data. The non-recurring cost for these courses is expected to be about Rs. 50,000 and the annual recurring cost, including the whole-time staff required is likely to be Rs. 30,000.

(d) That the State organization should supply to the Central Water and Power Commission for information and record, the scheme for measurement of structural action for each project, the specifications of instruments and equipment to be procured, the detailed installation instructions drawn up, and the periodic interpretation of results as embodied in the structural behaviour reports. This would provide the means for ensuring uniformity of practice in the country.

(e) That the Instrumentation Cell of the Central Water and Power Commission be strengthened to render it effective in :—

- (i) providing such information and assistance as may be required by State and other Design Organizations,
- (ii) formulation of the programme for progressive manufacture of instruments within the country,
- (iii) formulation training programmes for concerned personnel from each State,
- (iv) maintaining liaison with developments and activities outside India,

TERM OF REFERENCE-IV

- (v) drawing up standardization specifications for instruments and equipment,
- (vi) acting as a clearing house for foreign exchange applications in respect of instruments of all projects,
- (vii) reviewing proposals formulated by State Organization, the specifications drawn up, the installation procedures proposed, and
- (viii) reviewing the interpretation from results obtained.

This Instrumentation Cell should work in consultation with the Designs Directorate of the Central Water and Power Commission, and also with an advisory panel of engineers actively engaged in problems relating to the measurement and assessment of structural action.

5.0. TERM OF REFERENCE—V

“To recommend the most suitable method of procurement of instruments for various State Projects in the absence of indigenous products”.

Before making any recommendation it would be desirable to outline the present procedure for procurement of imported instruments.

Present Procedure

The project authorities send their applications, on the prescribed forms, to the Plant & Machinery Directorate of the Central Water and Power Commission, stating mainly; (i) Description of the instruments required, (ii) their cost and source of procurement, and (iii) the essentiality of the import. The Plant & Machinery Directorate simultaneously takes the following action:—

- (a) asks the Dams Directorate (Instrumentation Cell) to look into the essentiality of the import, and
- (b) applies to the Director-General, Technical Development for a ‘non-availability certificate’ from an indigenous angle.

The Central Water & Power Commission applies to the Union Ministry of Irrigation & Power for the release of foreign exchange in consultation with the Ministry of Finance and conveys the sanction to the Chief Controller of Imports, who, in turn, issues a license directly to the project authorities.

On completion of the above formalities, the project authority procures the required instruments either directly from abroad or through the manufacturer’s agents in India.

In the case of procurement from the United Kingdom or the United States of America, the import license is issued by the Chief Controller of Imports to the India Supply Missions accredited to those countries. The supplies are to be procured through these agencies.

It is reported that generally 1½ to 2 years lapse from the date of the application to the time of actual procurement.

Discussion of the Present Procedure

The problem of procurement of instruments from abroad has been discussed with the concerned officials of the Central Water and Power Commission and of the Technical Development Wing of the Ministry of Industries and Supplies. It appears to the Committee that the basic problem to procurement from abroad is the absence of specific allocation of funds in foreign exchange for the purpose. All requirements of foreign exchange for the importation of instruments have to

be met from the “free” resources of the country except for the loan-tied projects where the position is somewhat easier.

The Committee is of the opinion that instrumentation of dams and other structures for structural behaviour studies is of sufficient importance so as to warrant pre-allocation of funds in foreign exchange, for the importation of essential instruments for at least the recommended obligatory measurements.

The procedure involved in obtaining the non-availability certificate (from indigenous sources) from the office of Technical Development Wing is another difficulty in early processing of foreign exchange applications. At present, this certificate is required to be obtained in the case of each application for the import of instruments. As it has been brought out under term (vii), the Committee do not anticipate indigenous manufacture of a number of instruments for reasons explained therein. It is therefore desirable that the ‘non-availability’ certificates may be issued in respect of a particular instrument for a specific period. This will obviate the necessity of clearing each application individually for the import of that instrument through the Technical Development Wing. Having obtained this general clearance in respect of a particular instrument, the Central Water and Power Commission will be able to clear all applications for that instrument during the validity of the certificate.

At present the Plant and Machinery Directorate of the Central Water & Power Commission scrutinizes the applications for the sanction of foreign exchange. In most cases the Plant and Machinery Directorate forwards the applications to the Instrumentation Cell of the Commission for the review of the Essentiality Certificate given by the applicant. In the event a clarification is sought from the applicant by the Instrumentation Cell, the reference is again routed through the Plant & Machinery Directorate. This causes unnecessary delay.

It has been reported to the Committee that, in most cases, the essentiality of the import is not fully explained by the applicants and is rarely supported by the Instrumentation drawings pertaining to the project. This necessitates considerable unnecessary correspondence between the applicant and the authority scrutinizing the application.

Recommendations

In the light of the above discussion, the Committee recommends as follows:—

- (i) Assessment of foreign exchange required for instruments should be made for all the projects for the plan period.

TERM OF REFERENCE-V

- (ii) This assessment should be consolidated in the Central Water & Power Commission, Instrumentation Cell on the basis of data and estimates furnished by the project authorities.
- (iii) The review of the Technical Development Wing regarding non-availability of instruments from indigenous sources may be obtained in respect of the consolidated requirements prior to applying for pre-allocation of foreign exchange to the Ministry of Irrigation and Power and Ministry of Finance.
- (iv) Pre-allocation of foreign exchange for the Plan period for the consolidated requirements should be sought from the Ministry of Finance. Whereas the requirements of instruments for obligatory measurements can be assessed with a fair degree of certainty, the assessment of special instruments may need substantial modifications at the time of detailed designs. This would call for some flexibility in allocation in respect of special instruments although the overall estimate may not be materially affected.
- (v) Non-availability certificate be obtained from Technical Development Wing for each type of instrument included in the consolidated list and not for each application. Technical Development Wing may give this certificate for the maximum period possible. Proposals for obtaining such certificates will be initiated by Central Water and Power Commission, Instrumentation Cell.
- (vi) Individual application for import licence be submitted by Project authorities to the Central Water and Power Commission complete in all respects and supported by necessary drawings pertaining to the structure, one additional copy being furnished to the Instrumentation Cell of the Central Water and Power Commission to expedite the acceptance of essentiality certificate furnished by the applicant, and certification of non-availability from indigenous sources based on information at (v) above.

6.0. TERM OF REFERENCE—VI

“To examine the need for maintaining a stock of such instruments in the country, so that the requirements of projects can be properly met and instrumentation received due implementation without any setback to the construction programme.”

The Committee considers that requirements can be best met by advanced planning and procurement of necessary instruments for projects rather than by maintaining a central stock. The question of meeting these requirements in time so that instrumentation programmes are implemented without any set back to construction schedules has been dealt with under term of reference (v). The Committee, therefore, do not favour the setting up of a central store of instruments. Other reasons which have weighed with the Committee in this decision are:—

(a) a centralised procurement will inevitably bring

in certain inflexibility in the matter of the choice of the instruments,

(b) will inhibit the growth of “measurement consciousness” on the part of State project engineers, and

(c) the overhead charges of running a central store are likely to be high. These will have to be eventually passed on to the State projects.

Even though a centralized agency for procurement of instruments has not found favour with the Committee, it is considered necessary that small stock of instruments may be kept in the Central Water & Power Research Station for use in the State projects for special purposes covering unforeseen and research requirements. This stock could be augmented by surpluses from projects.

7.0. TERM OF REFERENCE—VII

“To formulate a programme for the progressive manufacture of these instruments indigenously.”

The question regarding indigenous manufacture of instruments has therefore been considered keeping in view :—

- (a) Committee's recommendations made under term (i) regarding the distinction between obligatory measurements and special measurements.
- (b) Assessment of future requirements of instruments up to the end of Fourth Plan, made under term (ii).
- (c) Committee's recommendations made under term (iii) regarding the types of instruments best suited for use in our projects.

The availability of instruments necessary for obligatory measurements is of greater significance than for the relatively smaller quantity required for special measurements.

Start has already been made for indigenous manufacture in respect of some of the instruments for measurements on earth dams and other structures, such as piezometric tips, cross-arms for horizontal and vertical displacement including torpedoes, PVC tubing, valves and fittings, SR-4 gauges, dial gauges, bourdon gauges and resistance thermometers, 3-Conductor cables, terminal boards and contacts etc. The Committee however feels that stricter quality control is necessary which calls for detailed specifications and facilities for testing and calibration. It is strongly recommended that the latter facilities are provided at a few selected Research Laboratories in the country. Some of such tests are :

Pressure gauges calibration, air entry and permeability of Ceramic stone pressure testing of valves and fittings, calibration of strain gauges etc.

Though there has been lately considerable emphasis on self-sufficiency in the country, the indigenous manufacture of measuring instruments has its limitations in this respect. The primary factors which has weighed with the Committee in recommending any instrument for indigenous manufacture are dependability of the instruments and the volume of future demands. There

are other important factors as well, such as the availability of the specialised skill, the worth of the parts required to be imported, existence of an allied industry, etc. Considering all these factors the Committee has categorised the recommended instruments on the basis given against each, with a view to assigning due priority in each case :—

Category I (High priority) in which (i) demand is sizable, (ii) used in other allied fields, (iii) involves extension of an existing skill, and (iv) may not require imported parts, or the imported components are small in value.

Instruments in this category are : Micrometer slide and microscope for deflection measuring equipment; Carlson test set; Vibrating wire test set; PVNC tubing; SR-4 gauges; Dial gauges; Bourdon gauges; extruded aluminium tubing for inclinometers.

Category II (Middle priority) in which (i) by importing special material of small worth, an allied industry may be able to manufacture the instrument; (ii) demand is not as high as under Category I, (iii) the required skill is not existing at all at present.

Instruments in this category are: Carlson strainmeter; Carlson jointmeter.

Category III (Low priority) in which (i) demand is very small (ii) use is specialised (iii) basic cost of the instrument is that of the imported components, and (iv) highly specialized skill is necessary.

Instruments in this category are : Carlson stressmeter; Carlson pore pressure cell; total earth pressure cell; contact earth pressure cells; Wilson inclinometers measuring set; Huggenberger strain gauge; Invar tape and rod; Wild T3 theodolite; Wild N3 Level.

In order to encourage the manufacturers of quality instruments in the country, preference should be given while making purchases, to manufacturers of proven performance. The usual requirement of accepting the offer of the lowest bidder cannot be strictly adhered to.

The Committee recommends that reputable manufacturers in India should be permitted to import sample instruments liberally from manufacturers abroad for the purpose of affecting improvements in their products.

APPENDICES

APPENDIX 0.01

(To be Published in the Gazette of India, Part I, Section I)

No. 20(31)/63-Policy

Government of India

Ministry of Irrigation & Power

New Delhi the 31st Oct., 1964

RESOLUTION

Various instruments are embedded in, or installed on the surface of dams for measuring the structural action in dams foundations, retaining walls, etc. Different aspects of such instrumentation deserve careful study, with a view to ensuring continuous scientific and up-to-date methods for measurement of structural action in dams and other hydraulic structures. The Government of India have decided to set up a Committee, with the following composition, for carrying out the study in detail after visits to the dams, where necessary :

1. Shri P. S. Bhatnagar,
Chief Design Engineer,
Bhakra & Beas Designs Orgn., Kakanagar, New Delhi. *Chairman*
2. Shri K. R. Datye,
Director,
Maharashtra Engineering Research Institute, Nasik. *Member*
3. Shri B. Pant,
Chief Research Officer,
Central Water & Power Research Station, Poona. *Member*
4. Shri I. P. Kapila,
Director Designs,
Bhakra & Beas Designs Organizations, Kakanagar, New Delhi. *Member*
5. Shri V. K. Sondhi,
Deputy Director,
Central Water & Power Commission. *Member-Secretary*

2. The terms of reference of the Committee will be as follows :

- (i) To make a general review of the instrumentation done so far in various dams in the country including general examination of data and results obtained therefrom, and their interpretation as indicative of structural action;
- (ii) To assess future requirements of instruments upto the end of Fourth Plan, with particular reference to the requests of the projects now under construction;
- (iii) To examine different types (such as Carlson elastic wire type, vibrating wire type etc.) of instruments and recommend the best suited for our projects;
- (iv) To recommend the best arrangements for recording storage, analysis and interpretation of field readings from instruments;
- (v) To recommend the most suitable method of procurement of instruments for various State projects in the absence of indigenous products;
- (vi) To examine the need for maintaining a stock of such instruments in the country, so that the requirements of projects can be properly met and instrumentation received due implementation without any setback to the construction programme; and
- (vii) To formulate a programme for the progressive manufacture of these instruments indigenously.

3. The Committee will submit its report by the end of June 1965.

Sd. M. K. KIDWAI,
31-10-64
Joint Secretary to the Govt. of India.

ORDER

Ordered that a copy of the Resolution be communicated to the Departments of all State Governments dealing with River Valley Projects and to the persons concerned.

Ordered also that the Resolution be published in the Gazette of India for general information.

Sd. M. K. KIDWAI,
Joint Secretary to the Govt. of India.

To

The Manager,
Government of India Press,
Faridabad.

Copy forwarded for information to

- (i) The Secretary to the Department of all State Governments dealing with River Valley Projects.
- (ii) The Central Water & Power Commission (10 spare copies).
- (iii) All Project Sections in the Ministry.

Sd. B. C. GANGOPADHYAY,
Deputy Secretary to the Govt. of India.

APPENDIX 0.02

COMMITTEE ON INSTRUMENTATION OF DAMS AND OTHER STRUCTURES

PART 'A'

Name of project and its location	Type of structure with brief description	Type of instruments installed, their number and location	Year of installation

PART 'B'

1. Name of Project.
2. Note on geology of site enclosing detailed layout drawings.
3. Cost and source of procurement of instruments.
4. Instrumentation drawings pertaining to the structure :
 - (i) Total period for which observation records are available.
 - (ii) Frequency of observations.
5. Present condition of instruments (operative or inoperative).
6. Analysis and interpretation done, if any.
7. Brief note on instruments used.

APPENDIX 0.03
CONCRETE DAMS

DAM & PROJECT		BHAKRA Bhakra Nangal Project, Punjab	KOYNA Koyna Hydro-electric Project, Maharashtra	RIHAND Rihand Dam Project, U.P.
DAM HEIGHT (Ft.)		740	338	304
LENGTH (Ft.)		1700	2650	3065
DISPLACEMENT	GEODETTIC METHOD	Theodolite Stations	No.	
		D/s Face of Dam	No.	47
			Year of Installation	1964
			Period of Observation	
		Abutments	No.	28
			Year of Installation	1964
			Period of Observation	
		Top of Dam	No.	15
			Year of Installation	1964
			Period of Observation	
		Galleries	No.	23
			Year of Installation	1963
			Period of Observation	
		Checking Points	No.	10
			Year of Installation	1964
	Period of Observation			
	Alignment (Collimation)	No.		
		Year of Installation		
		Period of Observation		
	Bench Marks (Level- ling Points)	Inside Dam Galleries	No.	52
			Year of Installation	1963
			Period of Observation	
		Abutments	No.	48
			Year of Installation	1964
			Period of Observation	
	PLUMB LINE	Regular	No.	5
			Year of Installation	1961-62
			Period of Observation	Since July 1961
		Reverse	No.	*
			Year of Installation	
			Period of Observation	
	JOINTMETER	Internal	No.	80 (4/76) Δ
			Year of Installation	1956-63
			Period of Observation	⊙
	Surface	No.		
		Year of Installation		
		Period of Observation		
	DEFORMATION MEASURING DEVICE	Foundation Deformeter	No.	
			Year of Installation	
			Period of Observation	
		Settlement Measuring Device	No.	*
			Year of Installation	
			Period of Observation	
	ROTATION METER	Clinometer	No.	
			Year of Installation	
Period of Observation				
Inclinometer		No.		
		Year of Installation		
		Period of Observation		
STRESS	STRESS METER	Carlson	No.	24(6/18) Δ
			Year of Installation	1956-57
			Period of Observation	⊙
	Vibrating Wire	No.		
		Year of Installation		
		Period of Observation		
STRAIN	STRAIN METER	Carlson	No.	418(14/404) Δ
			Year of Installation	1956-61
			Period of Observation	⊙
	Vibrating Wire	No.		
		Year of Installation		
		Period of Observation		
PORE PRESSURE	HYDROSTATIC PRESSURE CELLS	Carlson	No.	
			Year of Installation	
			Period of Observation	
	Vibrating Wire	No.		
		Year of Installation		
		Period of Observation		
TEMPERATURE	THERMOMETERS	Carlson	No.	178 (5/173) Δ
			Year of Installation	1957-62
			Period of Observation	⊙
		Thermocouple	No.	
			Year of Installation	
			Period of Observation	
Thermistor	No.			
	Year of Installation			
	Period of Observation			
Vibrating Wire	No.			
	Year of Installation			
	Period of Observation			
APPLIED LOAD	PRESSURE CELLS	Uplift Pressure Pipes	No.	50
			Year of Installation	1956-60
			Period of Observation	Since Sept. 1959
		Vibrating Wire	No.	
			Year of Installation	
			Period of Observation	
DYNAMIC CHARAC- TERISTICS	Seismographs	No.	2	
		Year of Installation	1964	
		Period of Observation	⊙	

* Installation of 1 No. reverse plumb line and 1 No. settlement measuring device awaiting procurement of special drilling equipment.
 ⊙ Observed since embedment to date and continuing except August 1959 to Feb. 1960 when instruments located at lower elevations could not be observed due to hoist chamber mishap.

□ Since embedment to date.

Δ Inoperative/operative.

APPENDIX 0.03—Contd.
STONE MASONRY DAMS

DAM & PROJECT		GANDHI SAGAR Chambal Valley Develop- ment, M. P.	HIRAKUD DAM Hirakud Dam Project, Orissa	LINGANAMAKKI Saravathi Project, Mysore	MALAMPUZHA RESER- VOIR Malampuzha Project, Kerala	NAGARJUNA Nagarjuna Sagar Dam Project, A. P.	PARAMBICULAM Parambikulam Aliyar Project, Madras	TUNGABHADRA Tungabhadra Dam Project, Mysore	WALAYAR RESERVOIR Walayar Project, Kerala		
DAM HEIGHT (FT.)		214	200	199		370		162			
LENGTH (FT.)		1685	3768	5740		4780		5712			
DISPLACEMENTS	GEODETC METHODS	Theodolite Stations	No.								
		D/s Face of Dam	No.	Year of Installation	Period of Observation						
Sighting Points (Targets)	Abutments		No.	Year of Installation	Period of Observation						
		Top of Dam	No.	Year of Installation	Period of Observation						
Galleries	No.		Year of Installation	Period of Observation							
	Checking Points	No.	Year of Installation	Period of Observation							
Alignment (Collimation)		No.	Year of Installation	Period of Observation							
	Bench Marks (Leveling Points)	Inside Dam Galleries	No.	Year of Installation	Period of Observation						
Abutments			No.	Year of Installation	Period of Observation						
	PLUMB LINE	Regular	No.	Year of Installation	Period of Observation	1					
Reverse			No.	Year of Installation	Period of Observation	2	1964				
	JOINT METER	Internal	No.	Year of Installation	Period of Observation	40(12/28)Δ					
Surface			No.	Year of Installation	Period of Observation	4	1962	14(1/13)Δ	6(6/0)Δ		
	DEFORMATION MEASURING DEVICE	Deformeter (Joint Meter)	No.	Year of Installation	Period of Observation						
Settlement Measuring Device			No.	Year of Installation	Period of Observation						
	ROTATION METER	Clinometer	No.	Year of Installation	Period of Observation						
Inclinometer			No.	Year of Installation	Period of Observation						
	STRESS	STRESS METER	Carlson	No.	Year of Installation	Period of Observation	20(7/13)Δ		10	14(8/6)Δ	4(0/4)Δ
Vibrating Wire				No.	Year of Installation	Period of Observation	1957-59		1962-64		1964
	STRAIN	STRAIN METER	Carlson	No.	Year of Installation	Period of Observation	10/10	55(2/28)Δ		28(6/22)Δ	14
Vibrating Wire				No.	Year of Installation	Period of Observation	1957-59	1954		Since 1959	Since 1956
	PORE PRESSURE	HYDROSTATIC PRESSURE CELLS	Carlson	No.	Year of Installation	Period of Observation	20(4/16)Δ			10(2/8)Δ	
Vibrating Wire				No.	Year of Installation	Period of Observation				Since 1960	
	TEMPERATURE	THERMOMETER	Carlson	No.	Year of Installation	Period of Observation	73(4/69)Δ	79(36/43)Δ	8	63(16/47)Δ	10(0/10)Δ
Thermocouple				No.	Year of Installation	Period of Observation	1957-59	1954	1962-64	1958-64	1964
	APPLIED LOAD	PRESSURE CELLS	Uplift Pressure Pipes	No.	Year of Installation	Period of Observation	20(0/20)Δ				
Vibrating Wire				No.	Year of Installation	Period of Observation					
	DYNAMIC CHARA- CTERIS- TICS	Seismographs	No.	Year of Installation	Period of Observation						
INFORMATION INCOMPLETE											
INFORMATION INCOMPLETE											

Notes : □ Since embedment to date.
Δ Inoperative/operative.

APPENDIX 0.03—Contd.
EARTH DAMS

8-3-C. W. & P. C/66

DAM AND PROJECT		BOR DAM BOR River Project, Maharashtra	BUGGAVAGU DAM Nagarjuna Sagar Project, A.P.	DANTIWADA Dantiwada Reservoir Project, Gujarat	DATLA DAM Datla Tank Project, M.P.	GANGAPUR DAM Gangapur Dam Project, Maharashtra	GHOD DAM Ghod Project, Maharashtra	GIRNA DAM Girna Project, Maharashtra	HATHMATI DAM Hathmati Reservoir Project, Gujarat	HIRAKUD DAM Hirakud Dam Project, Orissa	JIRGO DAM Jirgo Dam Project, U.P.	KOTAH BARRAGE Chambal Project, Rajasthan	LINGANAMAKKI Sharavathi Project, Mysore	MESHWA DAM Meshwa Reservoir Project, Gujarat	NALGANGA DAM Nalganga River Project, Maharashtra	NANAK SAGAR Nanak Sagar Dam Project, U.P.	SAMPNA DAM Sampna Dam Project, M.P.	UKAI DAM Ukai Dam Project, Gujarat			
DAM HEIGHT (Ft.)		119	100	120	87.5	123	94	172		195	90	120			98	54	76	228			
LENGTH (Ft.)		3800	8580	3882 Right 11643 Left	1340	12300	8738	1760		7000 Right 5000 Left			2160		8200		5700	13217.5			
DISPLACEMENT	Geodetic	Theodolite Stations	No.																		
		Sighting Points (Targets)	No.																		
			Year of Installation																		
			Period of Observation																		
		Bench Marks	No.																		
			Year of Installation																		
	Period of Observation																				
	Vertical by Internal Devices	Cross Arms	No.	1	19 (0/19)Δ	7	3			1	6 (1/5)Δ	1	2	3	1	1	1				
			Year of Installation	1965	1960-62	1960-64	1960	1954			1963	1952-56	1959	1959	1962-65	1964	1963	1960			
			Period of Observation		Since installed	Since installed		1954-59			Since 1964	Since 1952			Since Apr. 1962						
	Horizontal Displacement	Wilson Slope Indicator	No.																		
			Year of Installation																		
Period of Observation																					
Vibrating Wire Tiltmeter		No.																			
		Year of Installation																			
		Period of Observation																			
Electromagnetic Probe	No.																				
	Year of Installation																				
	Period of Observation																				
Plumb Line Well	No.																				
	Year of Installation																				
	Period of Observation																				
STRESS	Earth Pressure Cell	Carlson	No.																		
			Year of Installation																		
	Vibrating Wire	No.																			
		Year of Installation																			
PORE PRESSURE	Piezometers	Stand Pipe	No.							18		9									
			Year of Installation							1952-56		1960									
			Period of Observation							Since 1953											
	Hydraulic Tip Piezometer	No.	35	29 (0/29)Δ	66		91 (65/26)Δ	15 (7/8)Δ			116 (10/106)Δ		9		30 in 3 layers	25	7	22			
		Year of Installation	1965	1962	1960-65	1960	1954	1958		1963	1952-56		1963		1962	1960	1962	1963-64			
		Period of Observation					Since 1954	Apr. 60-Oct. 64		Since 1964	Since 1956		Since 1963		Since Sept. 62		Since 1962				
Electrical	Carlson	No.																			
		Year of Installation																			
		Period of Observation																			
V. Wire	No.																				
	Year of Installation																				

UNDER CONSTRUCTION

NOTE :- Δ Inoperative/operative.

APPENDIX 0.03—Concl.
OTHER STRUCTURES

PROJECT				TUNNELS & ROCK STRUCTURES			CONCRETE & PRESTRESSED CONCRETE STRUCTURES				
				Koyna Hydroelectric Project (Headrace Tunnel)	Koyna Hydel Project (Pressure Shafts)	Nagarjunasagar Project (Rt. Bank Canal)	Chambal Project (Kunu Syphon Aqueduct)	Koyna Project (Power House Foundation)	Vir Dam Spillway Piers	Sharavathi V. P. (Water Conductor System)	
LENGTH (Ft.)				12040	2066					10894	
DIAMETER (Ft.)				21	10 & 8' = 6"					36'—6" Wide	
LINING THICKNESS										20'—0" Height	
DISPLACEMENTS	Mechanical	Linear Changes	Invar Tape or Rods	No.							
				Year of Installation							
				Period of Observation							
			Dial Gauge & Mikrokators	No.			14				
				Year of Installation			1964-65				
				Period of Observation							
		Whittmore Huggenberger Gauge	No.								
			Year of Installation								
			Period of Observation								
		Angular Changes	Clinometers	No.							
				Year of Installation							
				Period of Observation							
	Precision Levels		No.								
			Year of Installation								
			Period of Observation								
	Optical	Theodolites & Precision Levels	No.				1				
			Year of Installation								
			Period of Observation								
Electrical	Displacement Pick-Ups	No.									
		Year of Installation									
		Period of Observation									
STRAINS	Whittmore Huggenberger	No.									
		Year of Installation									
		Period of Observation									
	Carlson	No.	6	14				2			
		Year of Installation	1961-62	1961-62							
		Period of Observation									
	Vibrating Wire	No.									
		Year of Installation									
		Period of Observation									
	SR-4 Gauges Including Concrete Gauges	No.			12	12		14			
		Year of Installation						1965			
		Period of Observation						3 Months			
STRESS	Dydic Gauge	No.									
		Year of Installation									
		Period of Observation									
	Carlson Stress Meter	No.									
		Year of Installation									
		Period of Observation									
Vibrating Wire	No.										
	Year of Installation										
	Period of Observation										
TEMPERATURE	Mercury Thermometer	No.									
		Year of Installation									
		Period of Observation									
	Resistance Thermometer	No.									
		Year of Installation									
		Period of Observation									
Thermocouples	No.										
	Year of Installation										
	Period of Observation										
Load Including Uplift	No.										
	Year of Installation										
	Period of Observation										

NOT YET STARTED

APPENDIX 0·04

(To be published in the Gazette of India, Part I)

No. 20(31)/63-Policy

Government of India

Ministry of Irrigation & Power

New Delhi the 6th July, 1965.

RESOLUTION

The period for the submission of the report by the Committee on Instrumentation of Dam set up under this Ministry Resolution No. 20(31)/63-Policy, dated the 31st October, 1964 is hereby extended up to the end of September 1965.

Sd. K. G. R. IYER,

Joint Secretary to the Government of India

Copy of letter No. 20(31)/63-Policy, dated the 6th July, 1965 from the Joint Secretary to the Government of India, Ministry of Irrigation & Power to the Secretary to Government of Punjab, Irrigation & Power Departments, Chandigarh.

ORDER

Ordered that a copy of the Resolution be communicated to the Departments of all the Governments dealing with River Valley Projects and to the persons concerned.

Ordered also that the Resolution be published in Gazette of India for general information.

APPENDIX 0·05

(To be published in the Gazette of India, Part I, Section I)

No. 20(31)/63-Policy
Government of India
Ministry of Irrigation & Power
Dated 29th September, 1965.

RESOLUTION

The period for the submission of the report by the Committee on Instrumentation of Dams, set up under this Ministry Resolution No. 20(31)/63-Policy, dated the 31st October, 1964, is hereby extended upto the end of December 1965.

Sd. K. G. R. IYER,

Joint Secretary to the Government of India

ORDER

Ordered that a copy of the Resolution be communicated to the Departments of all State Governments dealing with River Valley Projects and to the persons concerned.

Ordered also that the Resolution be published in the Gazette of India for general information.

Sd. K. G. R. IYER,

Joint Secretary to the Government of India

The Manager,
Government of India Press, Faridabad.

Copy forwarded for information to :

- (i) Secretary to the Department dealing with River Valley Projects in all State Governments.
- (ii) The C. W. & P. C., New Delhi (10 spare copies).
- (iii) All Project Sections in the Ministry.

Sd. N. N. KRISHNAN,

Under Secretary to the Government of India

APPENDIX I—1.01

Copy of letter from Prof. J. M. Raphael, Chairman, Division of Structural Engineering and Structural Mechanics, Department of Civil Engineering, University of California, Berkeley, Calif., dated February 23, 1965 to Mr. V. K. Sondhi, Deputy Director, Central Water & Power Commission of Govt. of India, Bikaner House, New Delhi, India.

Thank you for sending me a copy of the Govt. of India's Resolution of October 31, 1964, concerning instruments embedded in dams, with your letter of 12 February, 1965. I think you are to be commended for organizing a national committee on measurements. You will notice the word, "measurements", rather than "instruments", because I believe the emphasis should not be on hardware but rather on results. To elaborate on this statement, I have never felt pleased in making large installation of instruments in dams when subsequent results were not forthcoming. If your committee can influence the prosecution of the analyses of data from instruments already embedded in dams to the point where results of these studies are made available to your engineers, then it will have accomplished a very desirable thing indeed.

Another consequence of your formation of this committee is that you should endeavour to place a repre-

sentative on the International Commission on Large Dam Committee on Measurements. This should put your committee directly in contact with the thinking of other national committees on measurements, with immense value to you.

I commend you also on your choice of committee members. As you know, I am very familiar with the activities of the majority of the members of your committee, which I think you have organized very well. With regard to your specific question on the best type of instruments for dams, I am of course more familiar with Carlson elastic wire instruments than with the vibrating wire types. However, I have considered that the shapes of the vibrating wire instruments are such as to constitute maximum disturbance to the region of measurement and hence may themselves modify the very thing which is to be measured. I might also say that to my best knowledge the vast majority of complete and successful analyses that have been reported have been instrumented with the elastic wire devices. I should think that results should speak for themselves.

I am very happy to receive your letter and to know that you are still busy in this field and I hope that you will write again.

With my best personal regards,

APPENDIX I—1.02

Copy of letter from Mr. Roy W. [redacted] Civil Engineer, 55 Maryland Avenue, Berkeley, California 94707 dated 10-3-1965 to Shri V. K. Sondhi, Deputy Director, Central Water & Power Commission, Bikaner House, New Delhi, India.

Your letter telling of the formation of a committee for the study of instrumentation of dams was very interesting. I note that the committee has some well qualified members and should accomplish much.

Regarding the instrumentation of masonry dams, you have interpreted my views correctly; I do not recommend the use of stressmeters for masonry. And I suggest strain meters, only if they can be of long range so as to average out discontinuities inherent in a masonry structure. In fact, stresses and strains are difficult to interpret in any dam of low load stress, and especially difficult in the case of masonry. The

difficulty arises largely from the fact that extraneous influences such as temperature change and chemical action often obscure the smaller effects of load.

After reviewing some forty odd dams for safety, it has become evident that such reliance must be placed on precise surveys. Modern theodolites are so accurate that deflections of a dam and its abutments are measured with such accuracy as to give indication of any progressive changes which may be occurring. Targets on the dam and in the abutments will be the main instrumentation of some dams in Virginia.

What it comes down to is this; for dams where stresses are low, rely on survey points. Where stresses are high, then measurements of strain and/or stress are useful. However, where stresses are low, some useful results can be expected from long range, high-sensitivity strain meters or the equivalent.

With best regards.

APPENDIX I—2.01

REQUIREMENT OF INSTRUMENTS FOR CONCRETE/MASONRY DAMS UNDER FOURTH FIVE YEAR PLAN

Sl. No.	Name of the dam	Ht. of the dam in feet	Stress-meter for concrete dam only	Strainmeter		Joint-meter	Resistance thermometer		Pore pressure gauge	Deflection measuring equipment		Reading set
				For concrete dam	For masonry dam		Surface thermometer	Embedded thermometer		Plumb line	Micro-meter slide & microscope	
1	2	3	4	5	6	7	8	9	10	11	12	13
1	Vamsadhara A.P.	114					9			3	1	1
2	Pochampad A.P.	125					11			3	1	1
3	Mahi Stage-I (Weir) Gujarat	82					9	20		1	1	1
4	Ukai Project, Gujarat	215			90	57	18	34	5	3	1	1
5	Narmada (Broach) (Stage I), Gujarat.	220				57	18	34		3	1	1
6	Mahi Stage-II (Kadana), Gujarat.	251					22			3	1	1
7	Lidder Irrigation Project, J. & K.	210				57	18	34		3	1	1
8	Kallada, Kerala	216					18			3	1	1
9	Tawa Project M. P.	182					18			3	1	1
10	Parambikulam Aliyar											
	(a) Sholiar dam	319			90	75	26	47	15	3	1	1
	(b) Aliyar dam	71					13			1	1	1
	(c) Trimurthi dam, Madras.	74					13			1	1	1
11	Bhima Irrigation Project, Phagna Dam, Maharashtra.	119					9			1	1	1
12	Ghata Prabha Project, Mysore.	186			90	57	18	34	10	3	1	1
13	Upper Krishna Stage-I, Mysore.	145.5					13			2	1	1
14	Mala Prabha Project, Mysore	145					13			2	1	1
15	Harangi Project, Hudgur dam, Mysore.	120					11			1	1	1
16	Chawani H.E. Scheme, J&K.					42	9	20		1	1	1
17	Lower Jhelum H.E. Project, J. & K.					42	9	20		1	1	1
18	Sabargiri H.E. Project, Kerala											
	(a) Pamba dam	165				48	13	28		2	1	1
	(b) Kakki dam	346	8	102		28	10	18		1	1	1
19	Idikki H.E. Project, Kerala	550	25	549		111	42	76		3	1	2
20	Kuttiyadi H.E. Project, Kerala	129.5					13			2	1	1
21	Kotah Dam, Rajasthan	95				42	9	20		1	1	1
22	Vaitarna H.E. Scheme, Maharashtra.	154					13			2	1	1
TOTAL			33	651	270	616	375	385	30	52	25	26
					921			760				
Plus 20% for new projects in 4th Plan plus 10% for research : TOTAL 30%			10		276	185		228	9	16	8	8
			43		1197	801		988	39	68	33	34
			45		1200	800		1000	40	70	35	35

REPORT OF THE COMMITTEE ON INSTRUMENTATION FOR DAMS AND HYDRAULIC STRUCTURES

APPENDIX I—2·01—Contd.

REQUIREMENT OF THE INSTRUMENTS FOR VARIOUS EARTH DAMS UNDER FOURTH FIVE YEAR PLAN

Sl. No.	Name of the dam	Height of Dam	Length of Dam	Piezometer			Internal Movement			Foundation settlement (Base plates)	Surface settlement (Emb.)
				Twin tube hydraulic			Vertical cross arm	Horizontal cross arm	Slope indicator		
				Founda- tion type	Embank- ment type	Porous tube type					
1	2	3	4	5	6	7	8	9	10	11	12
1	Vamsadhara	60	5500	21	30	14	36	11	28
2	Pochampad	..	4300	36	54	14	66	83	207
3	Ukai Project	215	15060	78	210	20	138	30	5	26	65
4	Kallada	40	760	21	30	5	36	2	4
5	Barna Project	120	1100	47	69	14	81	3	6
6	Tawa Project	63	120	14	108	90	224
7	Parambikulam Aliyar :										
	(i) Sholiar Dam	94	..	36	54	14	66	At spacing of	At spacing of
	(ii) Parambikulam	62	..	21	30	14	36	500 ft. along	200 ft. along
	(iii) Tunka Dam	66	..	21	30	14	36	length of dam.	length of dam.
	(iv) Peru Varipallam	72	..	21	30	14	36	Do.	Do.
	(v) Tekhadi	86	..	36	54	14	66	Do.	Do.
	(vi) Aliyar Dam	142	..	57	84	14	96	Do.	Do.
	(vii) Trimurthi Dam	70	..	21	30	14	36	Do.	Do.
8	Chittar Pattanamakal :										
	(i) Chittar Dam-I	72	..	21	30	14	36	6	14
	(ii) Chittar Dam-II	82	3720	36	54	14	66	8	19
9	Mula Dam	154	8750	57	84	20	96	15	5	18	44
10	Bag Project	75	3750	36	54	14	66	9	22
11	Itiadh Project	98	1975	36	54	14	66	4	10
12	Krishna Irrigation Project.:										
	(i) Dam at Dhow	160	17850	57	84	15	96	16	39
	(ii) Dam at Borkha	115	10964	36	54	15	66	22	55
	(iii) Dam at Konhar	155	6060	57	84	15	96	12	31
13	Bhima, Irrigation Project, Wani Dam.	137	7696	57	84	15	96	15	38
14	Jayakwadi Project, Stage-I	120	31680	36	54	20	66	72	162
15	Bhadra Project	..	3610	69	162	14	126	8	19
16	Ghatprabha Project	100	11400	36	54	15	66	23	59

APPENDICES

APPENDIX I—2.01

1	2	3	4	5	6	7	8	9	10	11	12
17	Upper Krishna (1)	..	32284	57	84	20	96	65	162
18	Kabbini Project	..	7752	69	162	14	126	16	39
19	Salandi Project	170	1000	63	120	14	108	6	14
20	Beas Unit II Pong Dam	350	5750	96	324	30	168	20	10	12	29
21	Ramganga Project :	412	1950	96	324	30	168	20	10	4	10
22	Kangsabati Project										
	(i) Kangsabati Dam	130	13200	57	84	14	96	27	68
	(ii) Kumari Dam	135	6420	57	84	14	96	14	35
23	Umiam Barapani Project	52	416	21	30	14	36	2	5
	Add 30% for projects not accounted for	1529	2889	514	2673	60	30	574	1406
		459	867	154	802	65*	9	172	422
				1988	3756	668	3475	125	39	746	1828
	Say			2000	3800	670	3500	125	40	750	1830

*Additional 65 are proposed for dam where contact pressure cells are provided.

REQUIREMENT OF INSTRUMENT FOR 'OTHER STRUCTURES' UNDER FOURTH FIVE YEAR PLAN

Project	MEASUREMENT												
	Deflections				Stress meters		Strain meters			Thermometers Resis- tance & Test sets			
	Theo- dolite T3	Level N3	Dial Gauge	Invar Tape	Settle- ment gauge	Inclino- meter	SR-4	Carl- son	Vibra- ting Wire	Huggen- berger	Thermo- couples		
NORTHERN ZONE	Tunnels & Rock Struct.	3	2	75	4	5000	10	10	2	..
	Concrete & Pre- stressed Conc. Struct.	1	2	150	..	10	3	8	15000	3	40
CENTRAL ZONE	Tunnels & Rock Struct.	2	1	50	4	5000	7	7	2	..
	Concrete & Pre- stressed Conc. Struct.	1	2	75	..	7	2	6	10000	3	30
SOUTHERN ZONE	Tunnels & Rock Struct.	2	1	50	4	5000	78	8	2	..
	Concrete & Pre- stressed Conc. Struct.	1	2	100	..	8	3	6	10000	3	30
	TOTAL	10	10	500	12	25	8	20	50000	25	25	25	100
													8

NOTE :-The estimate of the instruments is based on the recommendation of the Committee that one or two typical structures will be selected in each zone for intensive study.

APPENDIX I—3.01

WILDE T-3

PRECISION THEODOLITE FOR TRIANGULATION

1. Shock resistance (shock absorbing capacity)
2. Stability of adjustment
3. Mechanical parts to have the same dilatation coefficient, instrument to be immune to either high or low temperatures.

The glass circles are read in two diametrically opposite places, a feature which is indispensable in a triangulation theodolite. An optical micrometer of extremely sturdy design gives directly the arithmetic mean of two diametrically opposite circle readings. The same micrometer is used for both horizontal and vertical circle; a lateral inventor knob permits to switch from the former to the latter. The microscope eyepiece lies alongside the telescope eyepiece, which allow to change over quickly from sighting to reading.

The purely dioptrical telescope has a free objective diameter of 60 mm (2.36 in.) and three interchangeable eyepieces for 24x, 30x, 40x magnifying power. The diameter of the 40x eyepiece exit pupil is still 15 mm (0.59 in.) wide, which fact and the antiglare treatment of the lenses produce extremely brilliant pictures, otherwise only to be obtained with mirror-type telescopes. Compared to the latter, T-3 telescope has a better constancy of the line of sight.

The two circles are usually illuminated with two double motion mirrors. However, the instrument can also be electrically illuminated. A socket has been provided in the instrument's base while hidden cables connect this socket with the two light sources. When introducing the lamps into the mirror sockets, the electric contact is automatically established. It supplies to connect the plug with battery case to light both bulbs which illuminate the circles and the telescope reticle.

The Wilde T-3 theodolite is mainly used in triangulation. The vertical circle has, therefore, been numbered in such a manner as to permit to compute the angle of height B from the difference between the circle readings A_1 and A_{11} in both telescope positions ($B = A_1 - A_{11}$). Transformation and computation of the mean value, indispensable otherwise, can thus be avoided.

For astronomical determinations of time and latitude a prismatic astrolabe attachment may be placed over the telescope's objective mounting.

Technical Data

1. Glass Circles 360° N 400 g
2. Horizontal Circle
 - (i) Diameter 5.9 in.
 - (ii) Graduation interval 4' or 10c
3. Vertical Circle
 - (i) Diameter 3.7 in.
 - (ii) Graduation interval 8' or 20c
4. Optical Micrometer
 - (i) Interval 0.2" or 1cc
5. Telescope
 - (i) Length 10.2 in.
 - (ii) Free objective aperture 2.4 in.
 - (iii) Magnification 24, 30 and 40x
 - (iv) Shortest focusing distance 15.0 ft.
 - (v) Field of view at 1000 ft. 28 ft.
6. Plate Level
 - (i) Sensitivity 7" per mm.
7. Vertical Circle Level
 - (i) Sensitivity 12" per 2 mm.
 - (ii) Setting accuracy $\pm 0.2'$

Equipment

1. Wilde T-3 precision theodolite 360° or 400 g
wt. 24.2 lb (11.0 kg)
2. Metal container with carrying strap 8.3 lbs. (3.8 kg)
3. Carrying frame 4.0 lbs. (1.8 kg)
4. Tripod 4a with fixed legs 15.6 lbs. (7.1 kg)

Optional Accessories

1. Battery box.
2. Eyepiece prisms.
3. Sunglass for telescope eyepiece prism.
4. Sunglass for telescope eyepiece.
5. Roe lofs of solar prism.
6. Prismatic astrolobe.
7. Polar attachment.
8. Optical plummet.
9. Pillar plate with centring pin.
10. Ball type centring for deformation measurement with accessories.

APPENDIX I-3.02

WILDE N3 PRECISION LEVEL

This instrument is ideal for high precision levelling, measuring power dam deformations, or bridges, and to check on the installation of large machines.

Telescope magnification is 42x. It can be focused down to a distance of 2m. The level sensitivity is 6" (sex) per 2 mm. In order to ensure the greatest possible stability of adjustment, the telescope body and level carrier are cast in one piece. A circular level is used for the overall levelling of the instrument, and it is centered by means of the three footscrews. The setting knobs, etc. are arranged in such a manner as to be easy to see and convenient to operate. Focusing on the reticle is done by rotating the eyepiece, and the target is focused by rotating the knob on the telescope right hand side. The telescope, which is free to rotate about the vertical axis, is locked with the azimuth clamp after coarse aiming, and the fine adjustment screw then is operated for perfect alignment. The eyepiece, in which the split bubble image is observed is immediately next to the telescope. The level bubble is shown in two half pictures which are brought to sharp coincidence by means of the vertical fine adjustment screw. This makes for very high precision in sighting.

The precision levelling rod with graduations on Invar should be used when highly accurate measurements have to be taken. Invar, as is well known has an extremely low expansion coefficient. A 3m long Invar tape is attached to the base of the wooden, box-shaped levelling rod and is maintained under fixed tension by a helical spring which connects the tape to the other end. The tape carries two precision centimetre scales which are shifted relative to each other, with

the corresponding digits seen on the wooden frame. One set of numerals gives the effective height over the aiming point, while the other one is shifted relative to it by a full 3 metres. This arrangement makes for a double independent reading, thus for refinement in the measurements, while guarding against gross errors.

The telescope reticle carries cross lines, the one horizontal arm of which consists of two hair lines which diverge slightly from the centre, so that they form a pointed wedge. The horizontal sighting falls between two centimetre divisions, and it is then optically shifted, until it coincides exactly with the nearest centimetre mark. This is where the wedge lines come in, since they are placed in the sharpest possible symmetry with respect to the centimetre graduation. This shifting of the line of sight is achieved by inclining an optical flat mounted in front of the objective, by rotating the micrometer head. In a third eyepiece, located below the level eyepiece, it is then possible to read the centimetre fractions on a brightly illuminated glass scale, down to 0.1 mm, with the possibility of making estimates to 0.02 mm.

The precision of such readings and overall accuracy is dependent on the manner in which the level is set up, the observer's pointing, and the optical micrometer. For a well-trained observer, the instrument alone will readily give maximum errors of about 1/4 of a second. Practically, over 1 km in both directions, the error can be reduced to ± 0.4 mm in height. Two neighbouring points near the top of the power dam can be separated to ± 0.05 mm by a double levelling.

APPENDIX I-3.03

SPECIFICATIONS FOR MICROMETER SLIDE AND MICROSCOPE

Micrometer Slide

This should be such that it can slide over a micrometer carriage (bracket) 13/18" wide on top, constructed in such a manner that it slides over the stainless steel carriage rod 0.997" \pm 0.003" in diameter, and anchored by cone-point screws seating into the drilled recesses in the rod. Graduations of the lead screw knob shall provide for readings to the nearest half thousandth of an inch. The carriage shall have a total range of travel of two inches.

Micrometer slide in use at Bhakra is M-301A type manufactured by Gaertner Scientific Corporation, Illinois, U.S.A.

Microscope

Microscope shall be 22 power containing fine cross hairs in the eye piece and focusing through 1½ inches and shall be of size so that it can be mounted in position on the carriage inside the observation recess, the recess being 24-inches high having wall faces parallel and normal to the dam axis, 12 inches from the centre line of the plumb line well and the front face (fitted with air-tight steel door) oriented to be at 45 degree to the wall faces, and at 10 inches from the centre line of the plumb line well.

Microscope in use at Bhakra Dam is M-101A type manufactured by Gaertner Scientific Corporation Illinois, U.S.A.

APPENDIX I-3.04

TYPICAL SPECIFICATIONS FOR HUGGENBERGER WHITTEMORE GAGE

General Description

Extensometers should employ two or more dial indicators mounted in a frame which should be attached to the test piece. A second frame should similarly be mounted at a set gage length (usually 8") and their relative motion measured as the strain. The frames should be secured to the specimen by means of pointed screws or "knife-edges" contacts. Reasonable accuracy in measuring unit strains should be obtained by using 0.0001" indicator, because of the relatively large gage length.

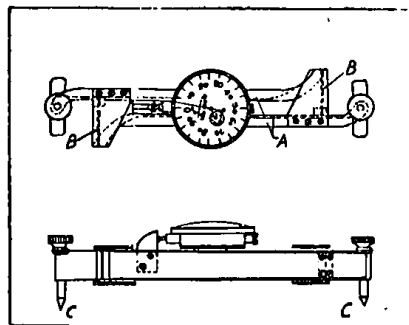
Gauges mounted in this manner should not be used for measuring plastic strains because the rapid decrease in diameter, which accompanies, yielding of the specimen, results in loosening of at least one of the frames. This suggests the desirability of a spring force in mounting instruments for measurement of plastic deformation.

The Whittemore Gage should be self contained instrument consisting essentially of two frame members (A) bound together by two elastic hinges, (B) for parallel frictionless motion. One 45° conical contact point (C) should be attached to each frame member. For strain measurements the contact points should be inserted into drilled holes defining a predetermined gage length. Motion between the two frame members, or strain, should be measured directly with a dial indicator. In another model the frames (A) should have been replaced by two co-axial tubes connected with a pair of elastic hinges, and, since the gage should be intended for repeated measurement at a series of stations rather than for fixed mounting at one station. Consideration should be given to controlling accidental longitudinal forces which might be applied by the operator. A handle serving doubly as a shield against temperature change and as an aid to uniform seating of the points should be attached to the gage by means of two elastic hinges. These hinges prevent application of excessive longitudinal forces. A force of about 5 lbs should be recommended for properly seating the points in the drilled holes. The holes should be reamed slightly to an angle of 60°, providing a reasonably definite line of contact with the 45° gage points.

Seating the gage is one of the chief sources of error. Other sources of error arise from the dial indicator, measurement of a chord rather than an arc length, and temperature changes. However, the effect of temperature changes should practically be eliminated by the use of an Invar tube for the inner tube of co-axial pair. The use of Invar necessitates a reference reading on a standard unstressed bar placed in the immediate vicinity of each station with each measurement on the station. Accuracies of 0.00002 unit strain should have been reported in field use of the 10 in. gage.

Manufacturer's Nominal Specifications

Gage length	.	2 in. 10 in. or to order (above 2" length).
Sensitivity	.	5 : 100,000.
Accuracy	.	0.0002 in. (approx.)
Range	.	0.10 in.
Size	.	1½ in. × 3½ in. × 15 in. (Approx.)
Weight	.	2 lbs. (Approx.)



THE WHITTEMORE
FULCRUM PLATE STRAIN GAUGE

APPENDIX I-3.05

CARLSON ELASTIC WIRE JOINTMETER

The Carlson Elastic Wire Jointmeter JM-10 is the standard jointmeter having a range of 0.20 inch used primarily for across a joint where it is desired to measure the joint opening at some point remote from the surface. The meter has the same indicating system as the Carlson Elastic Wire Strainmeter (Appendix I-3.15) with special construction features added to give it longer range i.e. by using single loops of steel wires held in tension by small steel coil spring instead of terminating the wires directly to the frames as in strainmeter.

One flange is replaced by a stud threaded to fix into the tapped end of a steel socket which is provided with each jointmeter for convenience in installation. The central bellow portion of the jointmeter is covered with cloth jacket to break bond with the concrete.

In addition to the rated elongation given below its construction has been detailed so that the meter can withstand a shearing movement of as much as 0.10 inch without damage or loss of calibration.

All jointmeters are supplied with 30 inches of 3-conductor rubber covered cable size 16 type/SO.

Dimensions and calibration data of JM-10

Overall length without socket	10 in.
Range, elongation, inches	0.20
Range contraction, inches	0.03
Weight with socket, lbs.	2.8
Least reading, inches	0.0005

APPENDIX I-3.06

SPECIFICATIONS FOR HUGGENBERGER TYPE OPTICAL CLINOMETER

The Huggenburger air bubble type clinometer essentially consists of a precision level which is a large radius vial containing sulphuric ether, and a rigid frame. The level mounted at mid span of the frame is pivoted at one end and fitted with a micrometer screw at the other. Measurements of the tilt are made by supporting each end of the frame on reference points attached to supports or bases embedded in the dam at desired location and in proper orientation and then centering the bubble by means of micrometer screw.

The clinometer base for these clinometers is made up of three stainless steel elements, one with a hole, another with slit and the third with a plane on which the clinometer supports rest. These bases have the number of elements required to measure rotation in two orthogonal planes. The distance between supports is about 10 cm. The sensitivity of the level is about 2".

Clinometers for different ranges of tilt are available.

APPENDIX I-3.07

SPECIFICATIONS FOR MAIHAK VIBRATING WIRE TYPE CLINOMETER

The measuring system comprises one receiver and any desired number of transmitters which are connected by cables to the receiver. The clinometer (transmitter) is permanently fitted to the object whose change of inclination is to be determined. The cylindrical case of the transmitter houses a special pendulum surrounded by damping oil with an extremely short pendulum length. A vibrating wire made of steel (the measuring wire) is stretched between the pendulum and the case. If the pendulum changes its position relative to the case, the mechanical tension and hence the natural frequency of the measuring wire is altered. An electromagnetic system picks up the mechanical vibrations of the wire converting them into electrical oscillations which are transmitted by cable to the receiver, which also contains a steel wire—the standard wire—whose natural frequency is tuned to the frequency of the measuring wire of the respective transmitter by suitable means until the required coincidence of frequencies exists.

Special measuring cable type GG, 2 × 1.5 mm should be used for installation in concrete, water and soil. The transmitters indicate the change of inclination in one measuring direction each. If the direction of inclination of the test object is not known or not constant, two transmitters are installed with their measuring directions offset by 90 degrees. The arrangement of the transmitters provides a direct measurement of the changes of inclination in the principal measuring directions.

Two types of clinometers MDS. 81 (with ball pin) and MDS. 82 are used depending upon whether the clinometer is to be mounted on the surface of the test object or to be embedded in concrete or fitted to buildings or construction parts exposed to hydraulic pressure respectively.

Manufacturer's Specifications for MDS. 81 (surface type) and MDS. 82 (embedded type) are as under :

MDS 81	a	b	c	82 a	b&c
Height, mm .	280	250	220	210	180
Diameter, mm .	95	95	95	210	210

Type	Measuring sensitivity smallest reading		Measuring Range
	When used with Receiver MDS. 3	When used with Receiver MDS. 4	
MDS 81 a, MDS 82 a . . .	2"	0.3"	±10'
MDS 81 b, MDS 82 b . . .	5"	0.7"	±20'
MDS 81 c, MDS 82 c . . .	10"	1.4"	±10'

Maximum measuring error of the complete measuring system smaller than 1% of the measuring range.

APPENDIX I-3.08

THE MILNE-SHAW SEISMOGRAPH (Tiltmeter)

(Make : Milne-Shaw Hilger & Watts Ltd., London).

This is a new type of seismograph with high magnification and electromagnetic damping. The magnification is approximately forty times greater than the standard Milne, while in practice the sensitivity to tilt is from ten to twenty times greater, according to the pendulum period adopted.

The general principle of the apparatus is to multiply the movements of a short damped boom by reflecting a beam of light from a pivoted lens of half-metre focus (seen in Plate 3).

The boom, about 19 inches long, carries a mass of 1 lb., together with a damping vane, which moves in a strong magnetic field, and brings the boom to rest after each excursion.

The outer end of the boom is coupled to and rotates the mirror, which is carried by a single iridium pivot in an agate cup, by this means 500 multiplications of the ground movement can be obtained, but in practice 150 to 250 multiplications are found to be more suitable. Two sets of coupling agate cups are provided, enabling either of the above multiplications to be adopted, as found desirable.

The mirror and coupling device are very light in weight; the friction from same has been found to be less than the air friction on the extension of the simple Milne boom.

The boom is damped by an electrolytic copper vane, fixed to its centre, floating between the poles of four adjustable alnico steel magnets, whereby any degree of damping up to a periodicity may be imposed.

The Milne-Shaw is not expensive to install, but its chief economy lies in its low consumption of paper. This has been rendered possible by an improved system of recording. The source of light (electricity, gas or oil) illuminates a vertical lens or slit and the image created is reflected and partially re-focused by the pivoted reflecting lens on to a second cylindrical lens placed horizontally, which completes the focusing of the light into a small intense point. This point falls with its centre upon a slit 0.003 inches wide; only the middle portion of the point is permitted to pass to the film, which is in immediate grazing contact with the slit. Perfect definition is produced in this way, with the result that waves of not more than two or three seconds period are shown quite distinctly on paper moving only 8 mm per minute.

Special calibrating and adjusting devices were necessary with such high magnification. This point has received special attention; tilts of 1/100 of a second of arc can be applied and registered by the image of a spider line on a distant calibrating scale. All such operations are performed, and the tilt and amplitude scales read, at a distance from the column: the motion being transmitted by a long flexible cable to the calibrating screw, so that there are no movements of the observer to enter into the amplitude shown in the trace. By this means the value in amplitude for one second of arc tilt may be determined within 1 or 2 per cent.

The re-adjustment of the light spot occasionally necessitated by variation in the daily tilt of the ground is performed by turning a milled head located near the recording box.

Artificial deflections of the boom for testing purposes are made by a solenoid near the boom, operated by an electric button at the recording box.

The record is timed by an electro-magnetic shutter encased within the lamp, which is to be connected in series with the observatory clock. No timing clock is included in the apparatus as supplied.

The recording mechanism runs for 30 hours.

The machine is not made twin-boom. Two separate units are used for recording the two components. These can be made right or left hand but are supplied right hand, unless otherwise ordered.

The type of illumination is optional and may be either, (1) a 4-volt electric lamp for which accumulators must be provided or a transformer with alternating current supply; (2) an incandescent bulb for ordinary lighting service (100-250 volts); (3) small incandescent gas mantle; (4) oil lamp. It is necessary to state which is required when ordering.

The Milne-Shaw machine, on account of its small mass and the minimum of inertia in its moving parts, is most sensitive to sudden changes of movement, enabling it to readily respond to the small waves of the P and S phases of a seismogram.

The machine is particularly suited for measuring small changes of level, such as deflection due to tidal load, etc. In general practice the machine is used at a comparatively low sensitivity to tilt; but no difficulty is experienced in obtaining very long pendulum periods of from 60 to 90 seconds, when the sensitivity to tilt

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becomes very great ; when set up to 90 seconds period with a multiplication of 250, 1 mm deflection in the trace is produced by so small a tilt as 1 inch in approximately 8,000 miles, or .004 seconds of arc = 1 mm amplitude.

Data

Length	5'6"
Width	2'6"
Height	2'1"
Length of boom	1' 6-7/8"
Boom point to mass	4"
Weight of mass	1 lb.
Distance between feet	11.459"
Size of drum	6" x 9"
Size of paper	19 1/2" x 9"
Speed of paper (per minute)	8 mm. or 16 mm.
Duration of run	24 hours or 12 hours
Damping	Electro-magnetic.
Damping ratio, suggested	20 : 1

Periods suggested	10 sec. or 12 sec. (adopted at Bhakra).
Magnification	150 or 250 or 350 (as adopted at Bhakra).
Calibration	One degree turn of screw produced one second of arc tilt.
Poising screw threads	50 per inch.
Working sensitivity	18 to 43 mm. per second of arc tilt, according to cons- tants adopted.
Illuminant	Electric lamp, in- candescent gas, or oil lamp, as or- dered.
Gas consumption	3/4 cubic ft. per hour.
Focal length of reflecting mirror	1/2 metre.
Weight when packed	2 cwt.
Size of packing case	36" x 27" x 22".

APPENDIX I-3.09

WILSON SLOPE INDICATOR

(Series 200)

(Manufacturer : The Slope Indicator Co. U.S.A.)

The slope indicator is a precision instrument for measuring the inclination of a nearly vertical surface. The instrument is lowered down a grooved plastic casing (or aluminium extruded casing) the grooves controlling the orientation of the instrument in a predetermined direction. Inclinations are taken at frequent interval of depth and are subsequently converted to displacements.

The slope indicator consists of a segment of precision-molded circular resistance element pivoted on instrument ball bearings with an eccentric weight mounted on the rim, enclosed in a water-tight aluminium cylinder 2-3/8 inches outside diameter and 15 inches long. Attached to the same inner frame as the pendulum, is a relay, which when activated by an electrical current, moves a conductor into contact with the resistance element and subdivides it into two resistances forming one-half of a conventional Wheatstone Bridge circuit. The other half, including switches and batteries, is enclosed in a control base at the ground surface. The instrument and control box are connected

by a heavy duty cable with yellow marker bands at one foot and red marker bands at five foot intervals. The instrument has two sets of guide wheels. Fixed guide wheels, aligned in a vertical plane parallel to the axis of the instruments, are mounted on one side while on the opposite side are a corresponding pair of spring mounted guide wheels.

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

The electrical supply is from four 6-volt dry cell batteries (RCA No. VS 317 Lantern battery or equal) wired in series. These batteries should be checked periodically and should be replaced when the voltage drops below 20 volts.

Slope indicator has a sensitivity of one part in 1,000 which means that it can detect a tilt of as little as three minutes of arc. This corresponds to a lateral movement of one half inch in 50 feet of depth. Displacements in thin shear zones of less than 1/16 inch can be easily identified.

APPENDIX I-3.10

TYPICAL SPECIFICATIONS FOR DIAL GAGES

The instrument incorporates patented method of construction which should enable a very high degree of accuracy to be maintained over the spindle travel of 0.15". The movement should be fitted with a jewelled bearings of the highest quality to ensure extreme sensitivity. Every effort should be made to produce an indicator of high accuracy yet of robust construction suitable for production and standards room use. With this object in view a shock absorbing mechanism should be built in to the instrument.

The indicator should be available in both english and metric readings, the dimensions should be identical to the standard range of "Mercer" Indicator Heads.

Diameter	Graduations	Readings	Travel	Magnification
2 1/4"	0-50-0	0.0001"	0.15"	598
2 1/4"	0-100	0.0001"	0.15"	598
2 1/4"	0-50-0	0.002 m/m	4 m/m	760
2 1/4"	0-100	0.002 m/m	4 m/m	760
3"	0-50-0	0.0001"	0.15"	765
3"	0-100	0.0001"	0.15"	765
3"	0-50-0	0.002 m/m	4 m/m	970
3"	0-100	0.002 m/m	4 m/m	970
4"	0-50-0	0.001"	0.15"	927
4"	0-100	0.001"	0.15"	927
4"	0-50-0	0.002 m/m	4 m/m	1175
4"	0-100	0.002 m/m	4 m/m	1175

APPENDIX I-3.11

TYPICAL SPECIFICATIONS FOR MIKROKATOR

The amplification unit should be a thin metal strip of rectangular cross-section (1) which should be twisted symmetrically on each side of the pointer attached to its middle and should hold at its ends under a tensile force. Variation of the tensile force should result in additional twisting or un-twisting of the two halves of the strip, with resultant rotation of the pointer. Within reasonable limits, the rotation should be proportional to the change in tensile force.

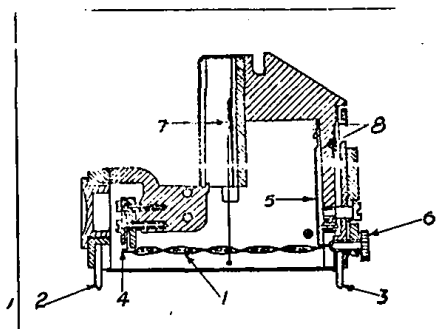
One end of the twisted strip (1) should be held by an adjustable bracket (4). The other end should be attached to a cantilever spring (5) which applies a continuous tensile force on the strip knife-edge (2) should be fixed while other edge (3) should be mounted to an assembly which should be constrained to move parallel to the direction of strain by the elastic hinges (8). Pointer (7) should be attached to the twisted strip and indicates strain values on a calibrated scale. Screw (6) should permit adjustment of the tension in the strip.

The instrument should be extremely sensitive and essentially frictionless and should require very little force to operate. Necessarily, its working parts should be delicate, although it should be represented as being protected sufficiently for outdoor exposure and as being sufficiently rugged for application to structures such as bridges. In any case its use in the Laboratory as a precision instrument for static strain measurement should only be limited by physical dimensions, difficulty of mounting or its weight. Normally the instrument should be held in place by means of light clamp-

ing force. The mounting clamp should have sufficient spring action to maintain mounting force despite dimensional changes.

Manufacturer's Specifications

	Most sensitive	Least sensitive
Gauge length	2"	2"
Least Graduations	0.00001"	0.0005"
Range	0.0005"	0.02"



THE JOHANSSON MIKROKATOR

1. Twisted strip
2. Knife edge
3. Knife edge
4. Adjustable bracket
5. Cantilever spring
6. Screw
7. Pointer
8. Elastic hinges

APPENDIX I-3.12

SPECIFICATIONS FOR CARLSON ELASTIC WIRE STRESS METER

The Carlson stressmeter (PC-800) with a range of 800 psi which is used in concrete consists essentially of a 7/8 inch diameter diaphragm with a strainmeter measuring unit mounted on one side. The diaphragm contains a mercury film and the stressmeter is designed to measure the compressive stress in the mercury film which is at all times equal to the stress perpendicular to the face of the diaphragm. The strainmeter unit is covered with braided fabric to break the bond with the concrete. Each meter is provided with 30 inches

of 3-conductor rubber covered cable, for splicing to longer cables if necessary. Calibration Data PC-800 Stressmeter.

Range psi compression	800
Least reading stress	5
Least reading temperature	0.1°F
Weight, pounds	6

APPENDIX I-3.13
SPECIFICATIONS FOR VIBRATING WIRE STRESSMETER

The specifications given hereunder are for Galileo type stressmeter.

The stressmeter essentially consists in the form of a capsule consisting of a base plate of steel to which the upper plate is secured. The two plates are connected to each other along the circular profile. The latter has been designed so as to act as a supported plate and it is stiffly locked to the base plate. Between the centres of two plates, the vibrating chord is stretched to detect deflections in functions of the load. The electro-acoustic detecting unit is of the normal type consisting of a vibrating wire and electromagnetic unit, suitable for connection to Galileo electro-acoustic control board.

The coil of the electro-acoustic detector is also used for temperature measurements. A three-pole cable protrudes from the capsule for the connection to the electro-acoustic central board and to the Wheatstone bridge for temperature measurements.

The main features of the instrument are as follows:—

Diameter	239·8 mm
Height (h)	110 mm
Weight	About 4 kg.
Area	452 cm ²
Measurement range	Upto 50 kilos/cm ² i.e. load upto 22,000 kilos.
Measurement Sensitivity	0·2 kilos/cm ² for each graduation (i.e. 80 kilos load).

Capsule deflection with 50 kilos load about 0·04 mm.

The capsule is supplied with a calibration diagram (the electro-acoustic central board graduations in function of the load) or with the calibration constant (central board graduations for kg/cm²).

The measurement chord is adjusted on the high values of the central board scale (460 graduations) so as to grant the whole measurement range. Upper screw plug allows any required chord tension adjustment. Inside of the capsule is placed a silica-gel drier to avoid condensed water effects of the internal air.

APPENDIX I-3.14

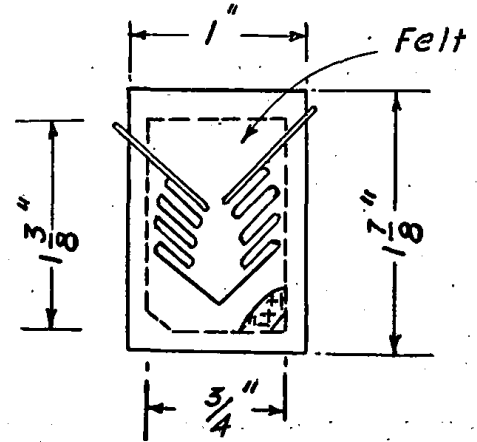
TYPICAL SPECIFICATIONS FOR DYDIC GAUGE

The strain-sensitive wire should be placed along one diagonal and an equal amount in series with it is placed along the other diagonal to cancel the effect of shearing strain on the deformation of the wire. This should be accomplished in the commercial stress gauge by winding the wire.

Type	Nomi- nal resis- tance in ohms	Gage factor $\frac{\Delta R}{\Delta L}$ R / L	Nomi- nal gage len- gth	Min. trim width	Fig.	Notes (see below)
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CV.28-3	500	2.6	—	3/4	—	1
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(1). NOTE:—Thin paper base (for rapid solvent elimination).



APPENDIX I-3.15

SPECIFICATIONS FOR CARLSON ELASTIC WIRE STRAINMETER

Carlson SA-10 is the standard strainmeter suitable for embedment in mass concrete. It comes fitted with a tapped hole (SAE- $\frac{1}{4}$ inch) for attachment to the specified strainmeter spider in case spider is used. The SA-10 Strainmeter is designed especially for field installation and under rugged placement conditions, for this reason has a safety stop at the lower limit of its range to minimise chances of breakage during embedment and transportations. Each meter is provided with 30 inches of 3-conductor rubber covered cable for splicing to longer cables if necessary.

This meter is in the general form of a long cylinder with anchors on the end to engage the surrounding concrete. The cylinder is covered with a cloth jacket to break bond with the concrete. Within the flexible brass cover tube, a steel framework supports porcelain spools around which are wound under 100,000 psi tension two equal coils of very fine steel music wire 0.0025 inch in diameter. The instrument is designed to take advantage of two electrical properties of steel wire resistance varies directly with temperature and resistance varies directly with tension.

The two coils are so wound that when the ends are pulled apart the outer or expansion coil elongates and consequently increase in resistance, while the inner or contraction coil shortens with decrease in resistance.

The ratio of the resistance of the expansion coil divided by the resistance of the contraction coil, which at all times is very near unity, is used as a sensitive measure of length change in the strainmeter. Temperature is measured by taking the sum of the resistance of the expansion and contraction coils. To ensure that corrosion cannot occur the coils are encased in a corrugated brass tube which is completely filled with corrosion resistant oil. A separate sealing chamber is provided per terminating the rubber cover cables that connect the strainmeter to the terminal boxes in the galleries. Dimensions and calibration data of SA-10 Strainmeter are as under—

Range in expansion millionths (minimum)	400
Range in contraction millionths (minimum)	800
Least reading strain millimetre (approx.)	3.8
Least reading temperature	0.1°F
Gauge length	10"
Flange diameter	1.25"
Body diameter	1.15"
Tapped for spider	Yes
Safety stop	Yes
Weight	1.6 lbs.

APPENDIX I-3.16

SPECIFICATIONS FOR MAIHAK VIBRATING WIRE STRAINMETER

The vibrating wire type strainmeter designated as Transmitter MDS-53, essentially consists of an anti-corrosive protective water-proof cylindrical casing, with anchors near each end to engage the surrounding concrete, containing a pretensioned steel wire and an electromagnet.

When the wire is plucked it oscillates with damped natural vibration and an alternating current is induced in the magnet circuit. This signal has the same frequency as that of the vibrating wire.

Change in the length of the wire (strain) produces corresponding change in the natural frequency of the wire and hence in the frequency of the signal. This change of the wire frequency can be read off on the receiver, by an appropriate tuning in method and represents an indication of the changed strain in the object under measurement.

The strainmeter is connected by a special measuring cable Type GG, 2×1.5 mm to the receiver or central measuring station. The accuracy and reliability of

measurements is not affected by the fluctuation of electrical values such as voltage, current, resistance or capacity.

The cylindrical casing is rugged enough to withstand shocks during embedment.

For measurement of strains in the interior of concrete structures transmitter MDS-53 is used and has the following manufacturer's specifications:—

Gauge length = 5 cm

Sensitivity strain/sc. divn. = 3×10^{-6} cm/cm

The standard type of this transmitter is pressure-water tight up to 15 atmos.

For measurement of surface strain, Maihak transmitter MDS-56 is used. Manufacturer's specifications are as under:—

Gauge length = 11.4 cm

Sensitivity strain/sc. divn. = 2.5×10^{-6} cm/cm.
Protective casing for water proof installation as for MDS-53.

APPENDIX I-3.17

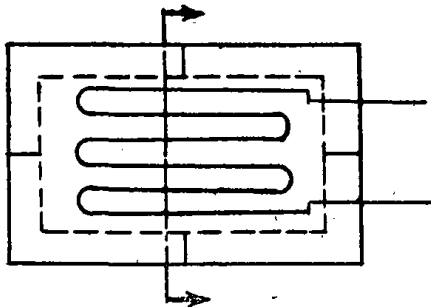
TYPICAL SPECIFICATIONS FOR SR-4 STRAIN GAUGE

Basically, the SR-4 Strain Gauge consists of about 5 in. of 1-mil wire cemented between two pieces of thin paper. The paper serves as a carrier for ease in handling the wire and also acts to insulate the wire from the metal surface on which it is to be bonded. The wire in the gauge is actually in the form of a grid consisting of a series of long, parallel loops (see Fig.) below.

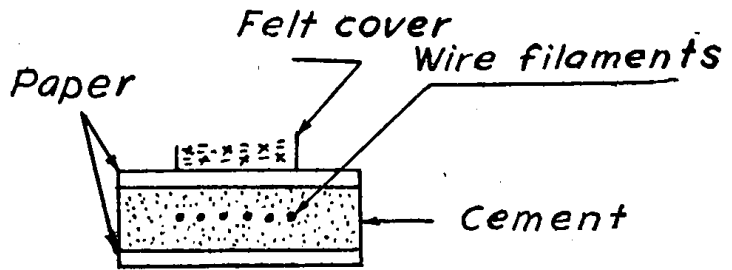
In the manufacturing process this grid is lowered into a piece of thin paper and fixed in place with a suitable cement, large diameter loads are then welded or soldered to the two ends of the grid wire, and a second piece of the paper is cemented over the wire. On some strain gauges a protective layer of felt is used to cover the top side of the gauge.

Dimensions of Typical SR-4 Gauge

Dimensions (inches)		Gauge factor	Resist- ance ohms	Output milli- volts per gauge	Temperature Max. deg. F.		Remarks
Length Gau- ge per	Wi- dth min.				Static	Dyna- mic	
0.062	0.56	0.12	1.7	60	2.5	‡ 175+200	For stress analysis of small fillets.



Flat Grid construction



Sectional view with vertical dimensions (Exaggerated)

APPENDIX I-3.18
SPECIFICATIONS FOR CONCRETE GAUGE

The principal components of a concrete gauge (or concrete deformation pick up) are a constant measuring wire, a temperature-compensation wire, a measuring cable and a screening tube. Measuring and compensation wires (SR-4 gauge type ref. Appendix I-3.17) connected in series and fixed to a paper support are enclosed in an hermetically sealed silver screening tube which protects the measuring element from moisture and the corrosive effect of weak concrete. The measuring cable connected to the element is a two core cable of extremely good electrical properties, highly resistant to chemical influences. The cores in a braided rubber jacket are surrounded by P. V. C. sleeving. As temperature influences strain gauge sensitivity, a compensatory element is built in. The pick up is built into a small concrete tile or rod the concrete having a composition of one part cement, two parts sand and three parts gravel (dry weight) the water cement ratio can vary between 0.6 to 1. The size of the gravel must not exceed 5 mm. This prevents mechanical damage to the pick up, permitting long term measurement in concrete.

<i>Technical Data</i>	
Resistance	120 ohms \pm 0.5 ohms.
Gauge factor (K-factor)	approx. 2.0
Tolerance of the K-factor	\pm 1.5%
at and tolerance of at	$-11 \pm$ micro strain / $^{\circ}$ C
Highest permissible temperature	60 $^{\circ}$ C
Measuring current	10 MA
Temperature compensation	From -10 to -40 $^{\circ}$ C max. deviation 1 micro strain / $^{\circ}$ C.
	40 to 60 $^{\circ}$ C max. deviation 1.5 micro strain / $^{\circ}$ C.
Modulus of elasticity of the pick up	Approx. 345 000kg/cm ²
Form factor	L/R-approx. 135
<i>Dimensions</i>	
Length of the measuring grid approx.	8 mm.
Length of the pick up approx.	200 mm.
Width approx.	12 mm.
Average thickness	0.2-0.4 mm.

APPENDIX I-3.19

SPECIFICATIONS FOR HYDRAULIC PIEZOMETERS

General Description

The hydraulic piezometer installation is comprised of twin-tube well by plastic tubing, filled with water. Pressures are transmitted from the piezometer tip through the water filled tubes and are observed on the gauges. The valves, pump, water supply tank, water filter unit, air trap and other accessories are installed in the terminal well to circulate water in the system and measure pore water pressures.

Material Specifications

Embankment and Foundation type piezometer tips shall be Bureau (USBR) type fabricated in accordance with details on U.S. Bureau of Reclamation drawing No. 40-D-5922 and 40-D-5819 respectively. The material specifications for components of the tips are as follows :—

(i) *Brass inserts for plastic tubing*—Brass inserts for 5/16 inch outside diameter by 1/16 inch wall thickness plastic tubing shall be equal to Parker fittings insert No. 5-T23 UI-B, Catalogue No. 4324.

(ii) *Keeper Plate*—1-3/4 inch dia×1/4 inch thick. This shall be of polypropylene plastic or hard board material.

(iii) *“O” Rings*—1-1/4 inch O.D. by 1 inch I.D.

These shall be of Nitrile rubber equal to Garlock Synthetic No. 24849—19: Garlock Inc. Denver, Colorado.

(iv) *Filter disc*—1 inch diameter by 1/4 inch thick.

The filter discs shall be made of a porcelain material equal to the P-3 porcelain manufactured by the Coors Porcelain Company, Golden, Colorado, having an average pore diameter of 1.3 to 2.0 microns. Tolerances on the filter discs shall be 0.005 inch for diameter and thicknesses of the disc, except that a break-off on corners and in the radius of the cutaway portion of the disc of 1/32 inch will be permitted.

(v) *Screws*—Screws shall be stainless steel, oval-head No. 4-40 (Conforming to American Standard) as shown in Drawing 40-D-5922.

OR

Screws shall be stainless steel slotted round head, dimensions conforming to 18×10, IS-1366-1962 and stainless steel material conforming to IS-1570-1961 grade 07 Cr., Ni., as shown on any of the piezometer tip drawings.

If the tip body is of brass material, the screws of the dimensions mentioned above shall also be of the same material.

(vi) *End Plate*—As shown on the drawing. The end plate shall be of stainless steel material either used by USBR or conforming to IS-1570-1961 grade 07 Cr., Ni. and of 18 gauge (0.050 inch) thickness. If the tip body is of brass material, the end plate shall also be of the same material.

(vii) *Tip Body*—1-3/4 inch diameter.

Tip body shall be of 1-3/4 inch diameter and of polypropylene plastic material or of brass material specified under ASTM 145 or any commercial quality available. The length of the body shall be as shown on the drawing No. 40-D-5922 or 40-D-5819.

(viii) *Compression Type Elbow 45°*—Required for joining 5/16" O.D. plastic tube to 1/8 inch Std. pipe. Compression type brass elbow 45° shall be of brass material, male type complete with nuts and ferrules; equal to Parker fittings Ridg-lok male elbow 45°, No. 5-VBU8-B-Catalogue No. 4323.

(ix) *Compression Type Connector*—Required for joining 5/16 inch O.D. tube to 1/8 inch pipe. The connector shall be of brass material; male type complete with nuts and ferrules; equal to Parker fittings: Ridg-lok male connector No. 5-FBU8-B Catalogue No. 4323.

(x) *Plastic Tubing*—Plastic tubing shall be of 5/16 inch outside diameter by 1/16 inch wall thickness and shall be manufactured from polyvinylidene chloride (PVNC) plastic material which shall meet the dimensional and average physical properties described below. The material of plastic tubing is the same as is used by USBR in the piezometer installations of earth dams. The plastic material used for extrusion of the tubing shall be homogeneous, virgin plastic and shall contain no foreign substances other than an inert colouring agent.

Dimensional tolerances—Tubing shall be conditioned for 24 hours at 73°F. and 50 per cent relative humidity. Average of six readings taken at different location on specimens.

Mean outside diameter . 0.315 inch, plus
0.005 inch, minus
0.010 inch.

Wall thickness . . . 0.060 inch, plus 0.005 inch
minus 0.005 inch. Cut section of tubing open lengthwise, use pointed head micrometer to reduce effect of curvature of tubing on measurements.

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Physical properties: Average of three specimens. Tests shall be conducted in accordance with ASTM Designations where referred to below:

Shore durometer (Type D), 65 minimum under initial application, 60 minimum after 15 seconds. Split tubing lengthwise, conduct test on stack of four split pieces of tubing ASTM Designation: D-1706-61.

Water absorption: 0.05 per cent after 24 hours immersion, maximum. Scrub specimens clean with water and cleaner, condition at 50 plus or minus 3°C for 24 hours. Wipe and blow dry after immersion. Specimens shall be 5 inches long ASTM Designation: D-570-59aT

Boiling water: Place specimens in pan of boiling distilled water for 5 minutes. Inspect for deterioration and whitening. Use same specimens for rupture strength test as described below. Specimens shall be 1 1/2 inches long.

Gas transmission: Average transmission rate of 1.0 cu. cm. per sq. m 24 hr. atmos., corrected to standard conditions, maximum. Test gas shall be carbon dioxide. Tests conducted on suitable flat specimens of 1.00 thickness, moulded from the same plastic material used in the tubing, test temperature 73°F. ASTM Designation: D-1434 (Proposed Revised 1963.)

Cold bend: Without collapse or rupture withstand a 360° bend around a 2-inch-diameter mandrel. Condition specimens and mandrel for 3 hours at 25°F.

and conduct test at 25°F. Bend tubing against the curve of the coil. Use same specimens for rupture strength test as described below. Specimens shall be 12 inches long.

Rupture strength: The rupture strength shall be determined for virgin specimens, boiling water specimens and cold bend specimens as prescribed below and in ASTM Designation: D-1599-62 T.

Tests shall be conducted in air at 73°F. Specimens load from zero to bursting in 60 to 75 seconds.

Virgin Specimens: 1,400 pounds per square inch, minimum, ultimate rupture strength Specimens conditioned 24 hours, in air, at 73°F. 50 per cent relative humidity.

Boiled specimens: 80 percent minimum of ultimate rupture strength. Specimens shall be cooled normally, in air, to 73°F.

Cold bend specimens: 90 per cent minimum of ultimate rupture strength. Specimens shall be warmed normally, in air, to 73°F.

Colour: Light yellow or amber. Semi-opaque tubing is desired, but is not required.

The ends of each coil of tubing shall be crimped or sealed prior to shipment. Tubing shall be shipped in 30-inch minimum diameter coils or spools, having a minimum of 1,000 feet of tubing per coil.

APPENDIX I-3.20

SPECIFICATIONS FOR CARLSON RESISTANCE THERMOMETER

Carlson Resistance Thermometer (TM-1) is especially designed and constructed for embedment in mass concrete. It consists of a coil of copper wire wound non-inductively on an insulating spool in such a way that there will be no appreciable-strain changes as the temperature changes. The entire active element is encased in a soldered brass case to prevent the entrance of moisture, and the element is further protected by filling the inside of the case with joint sealing com-

pound. All Carlson thermometers have a uniform resistance of 39.00 ohms at 0 degree F, and they change exactly 0.10 ohm per degree.

Each resistance thermometer is supplied with 30 inches of 3-conductor rubber cover cable size 16, type SO.

Range	0°F to 180°F
Least reading	0.10°F
Weight	0.5 lbs.

APPENDIX I-3.21

SPECIFICATIONS FOR THERMOCOUPLES

Thermocouple essentially consists of two dissimilar metal wires joined at ends to form an electrical circuit. Two pairs of thermocouple wire material widely used for the measurement of temperature in concrete are :—

- (i) Copper—Constantan : for use for low temperature range (-300° to 500°F) possesses relatively high thermo-electric power (average 23.8 micro volts per degree F in the range 32° to 212°F) and is resistance to rust and corrosion.
- (ii) Iron-Constantan : widely used for intermediate range (0 - 1000°F) has a nearly uniform and high thermo-electric power (30 micro volts per degree F in the range 32° to 212°F) mechanically strong and tough, and of moderately low thermal conductivity.

Thermocouple junction may be formed by joining the bared ends of the insulated conductors of the jacketed extension cable. Conductor wire may be either solid or stranded.

Wire size should be such as to provide adequate sensitivity, sufficient strength, flexibility and durability to withstand the rough handling during embedment operations and rugged exposure conditions. Wire size AWG 16 or 18 is preferable.

Insulations of the thermocouple element should be such as to provide positive and permanent protection against moisture penetration at the thermocouple junction and along the entire length of extension cable in order to reduce the possibility of spurious voltaic effects. Conductor insulation and sheath of cable to be embedded in concrete must be rubber (natural or synthetic) or polyvinyl plastic, preferably polyvinyl plastic under moderate temperature conditions (upto 175°F).

The wires should be available in quantities sufficient to avoid the necessity for making splices in any single thermocouple extension cable.

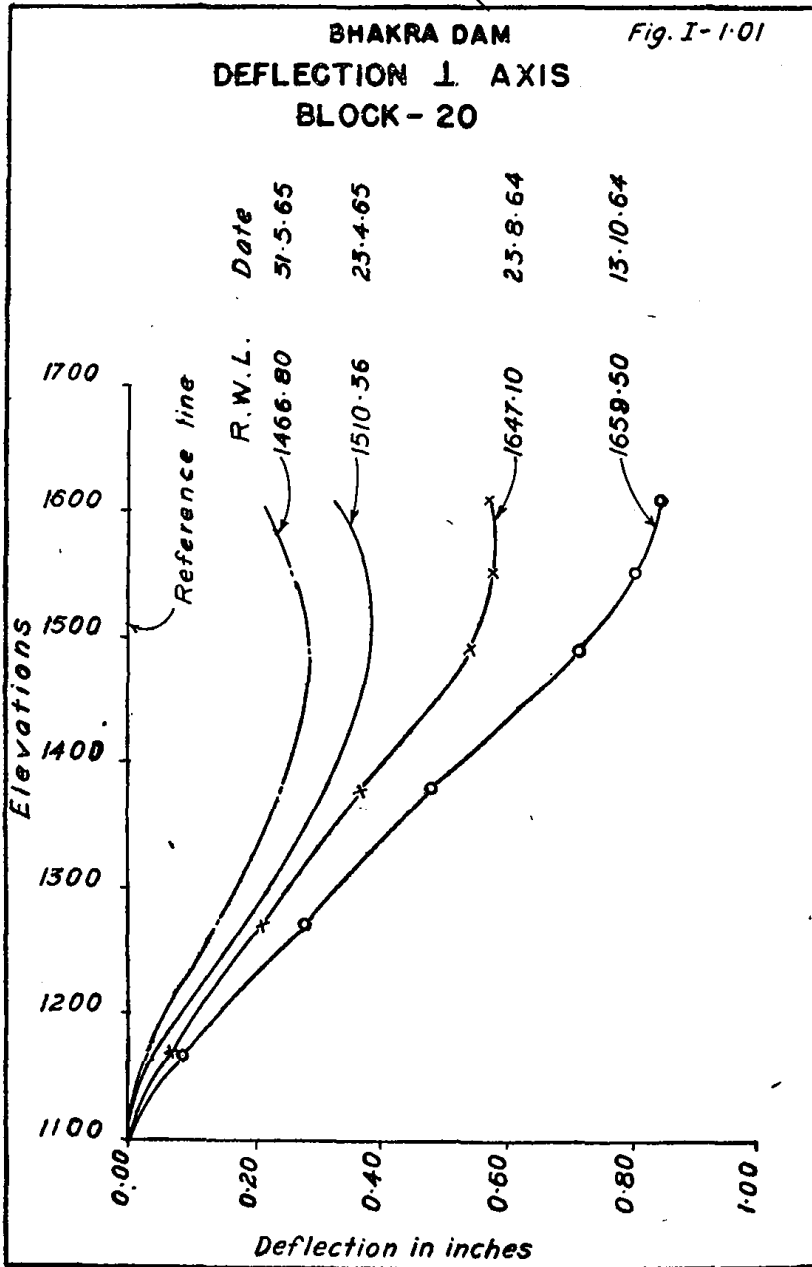


Fig. I-102

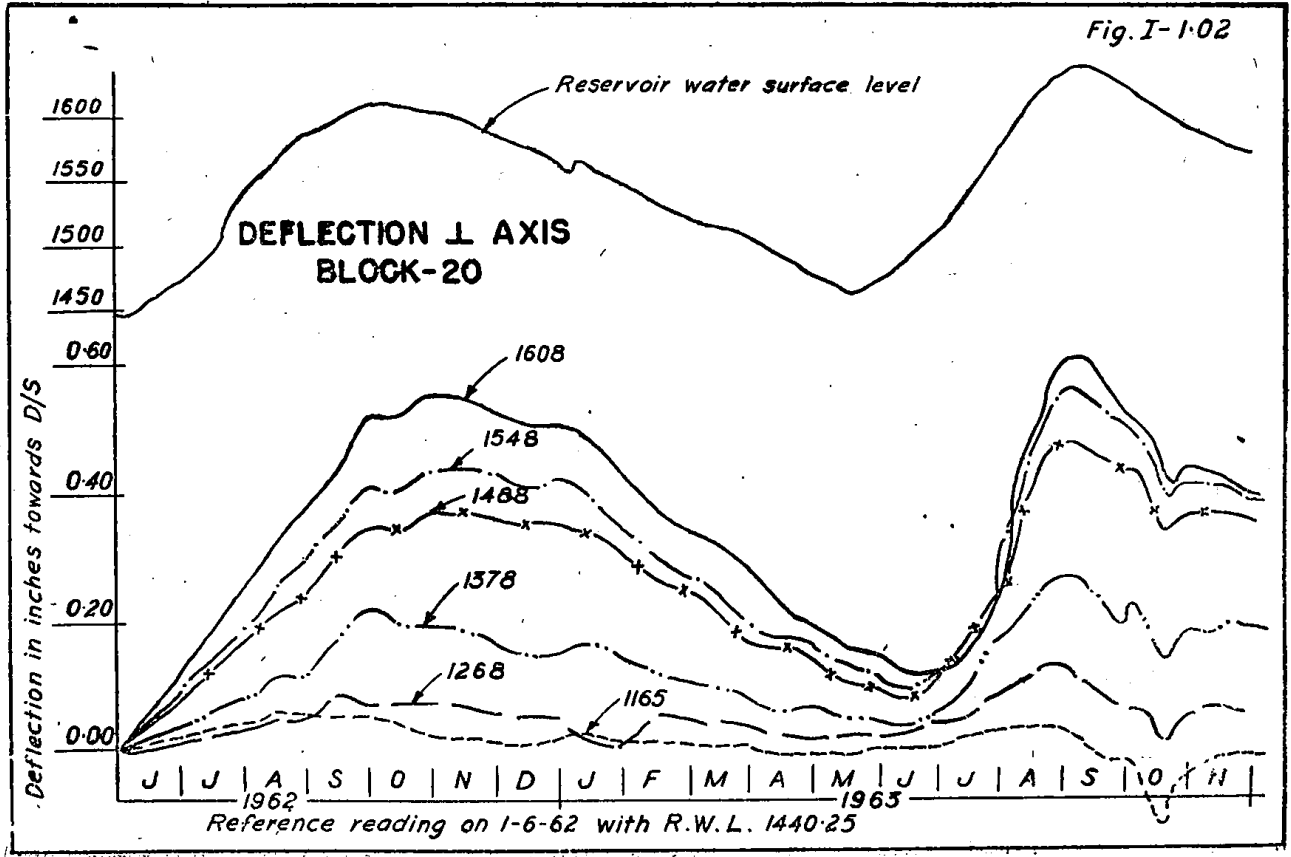


Fig. I-1-03

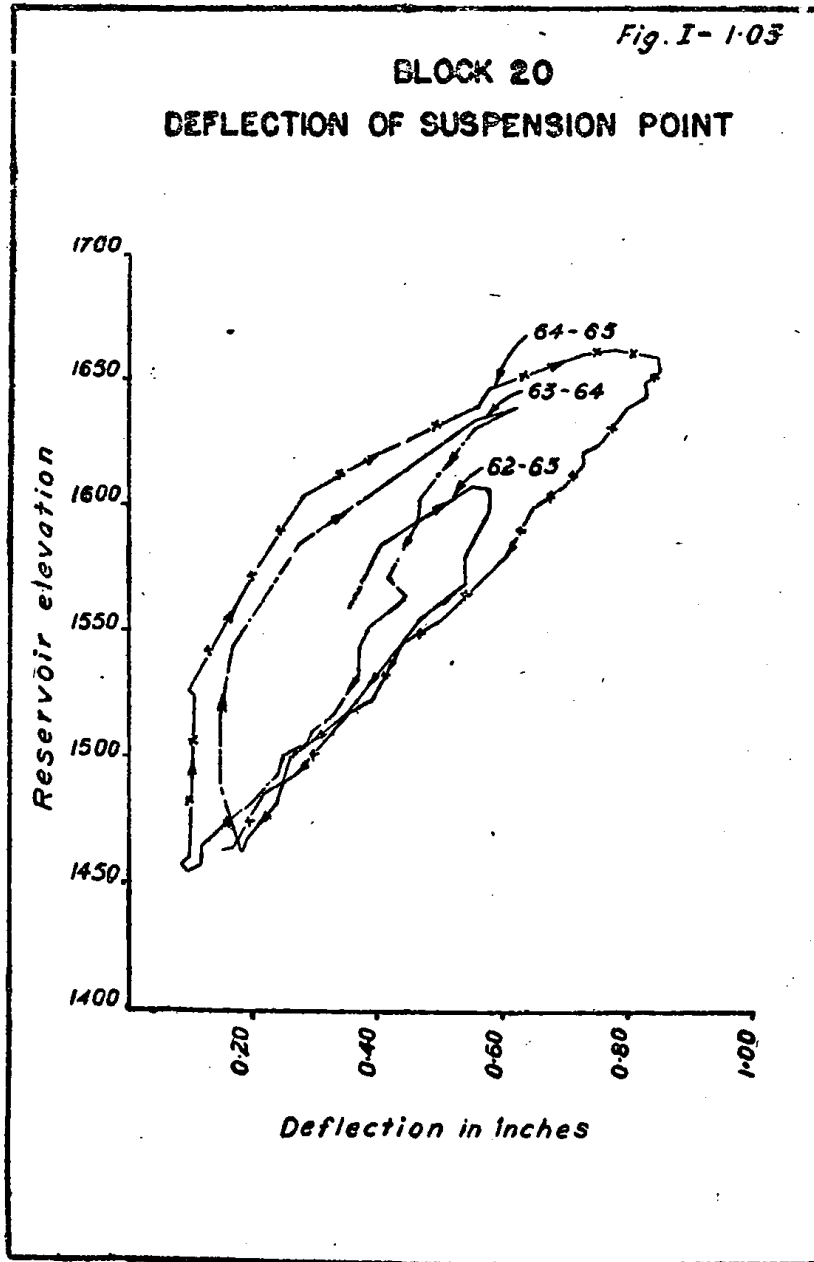


Fig. I-1-04

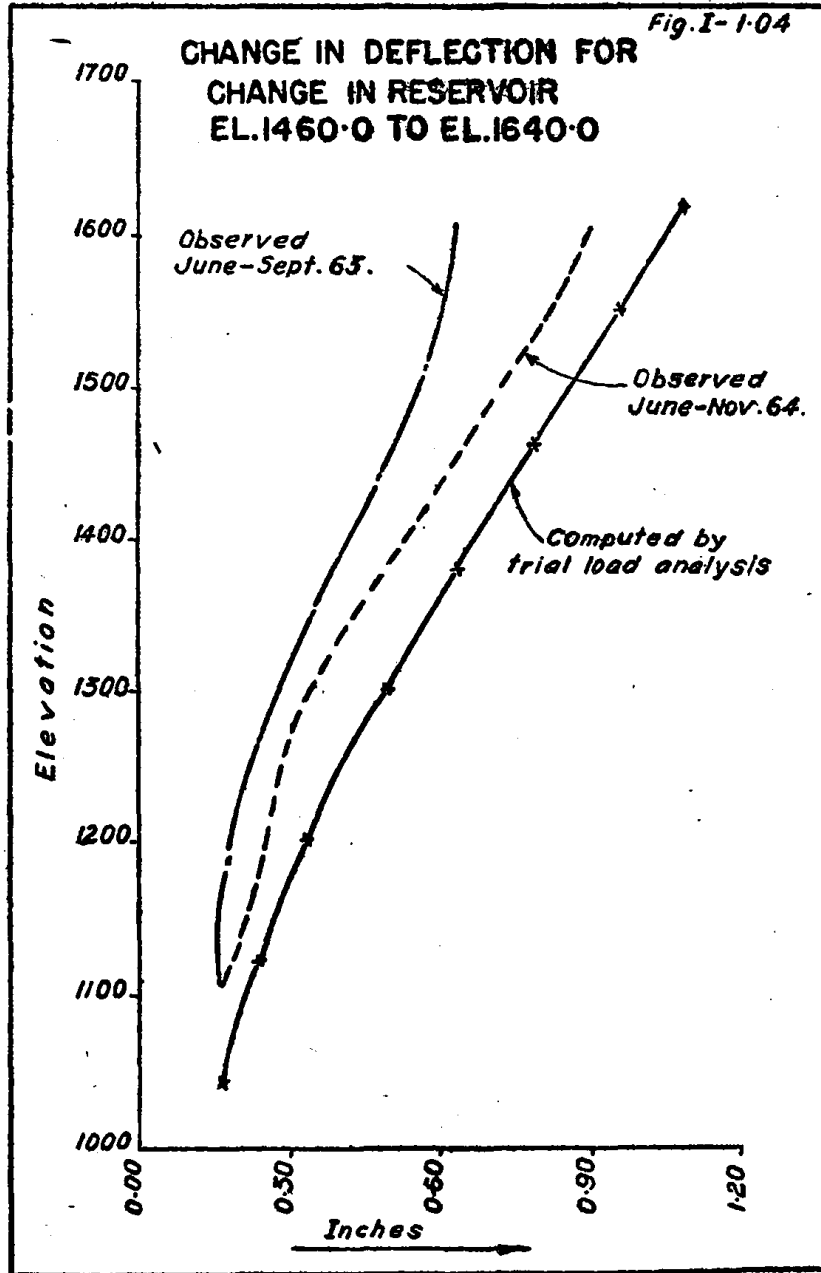
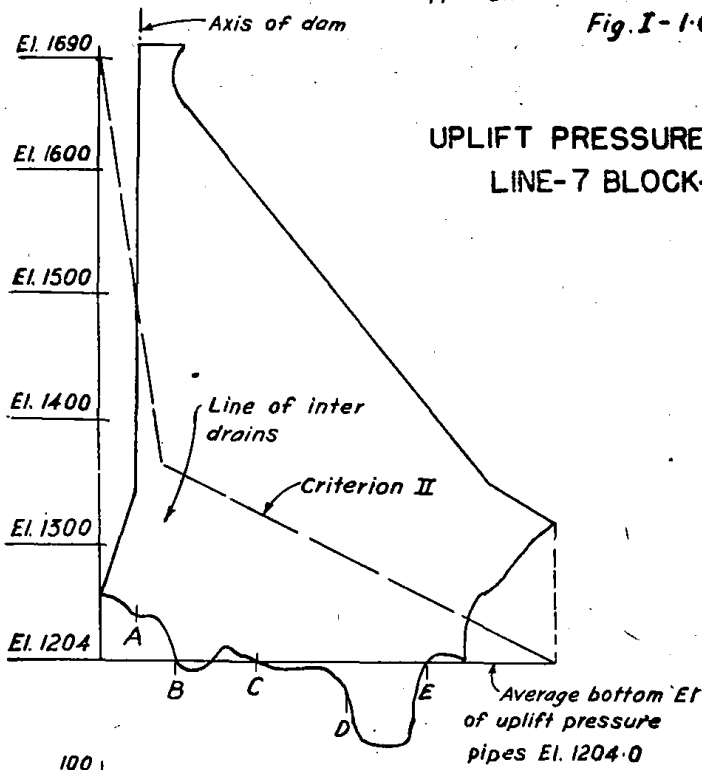
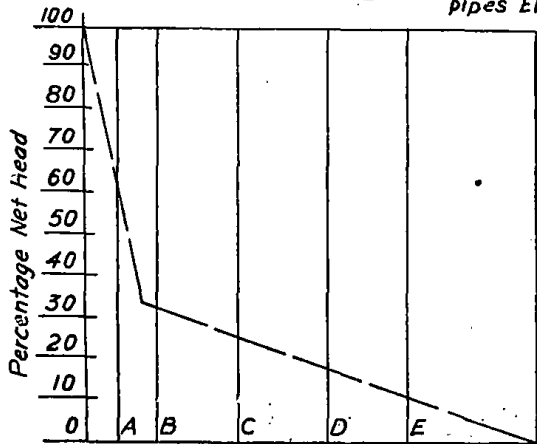


Fig. I-1-05



UPLIFT PRESSURE PIPES
LINE-7 BLOCK-25



LINE-7 BLOCK-25 STA. 16+07.00

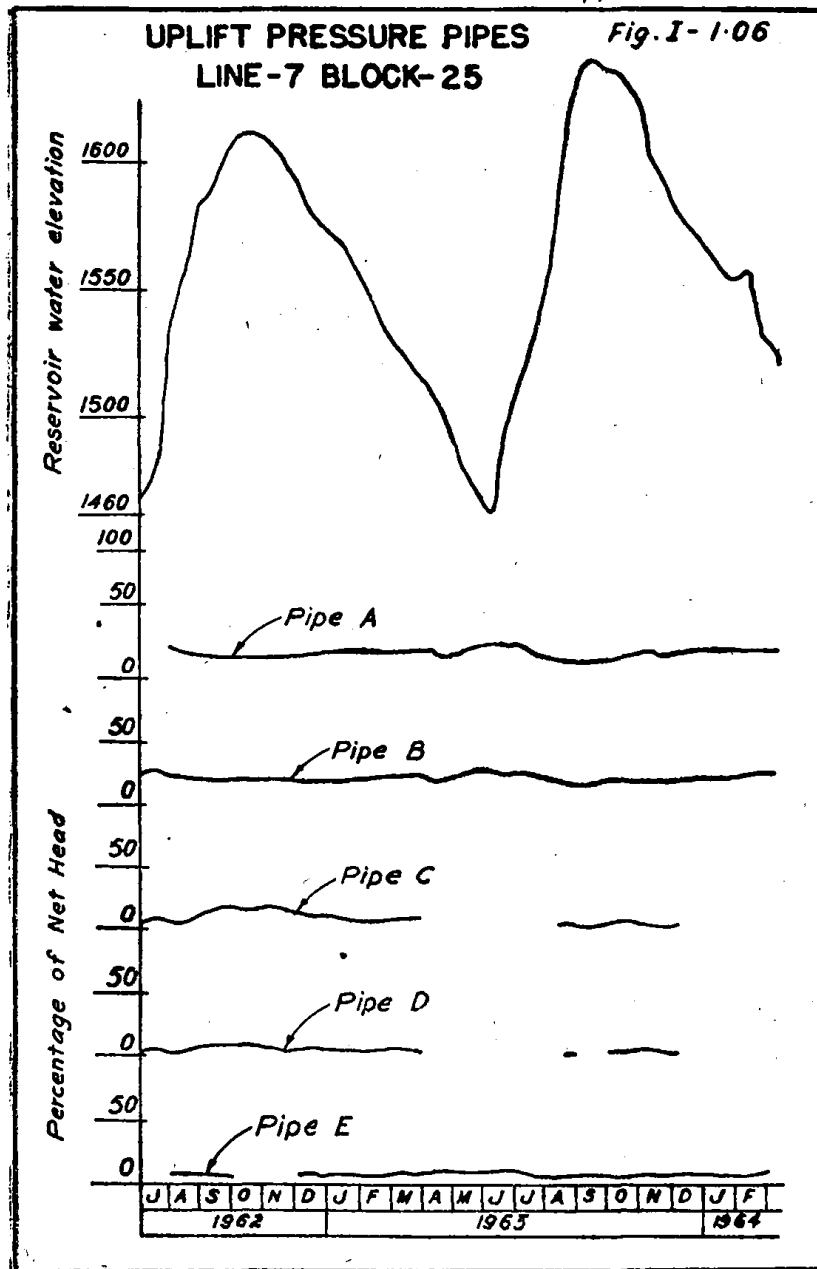
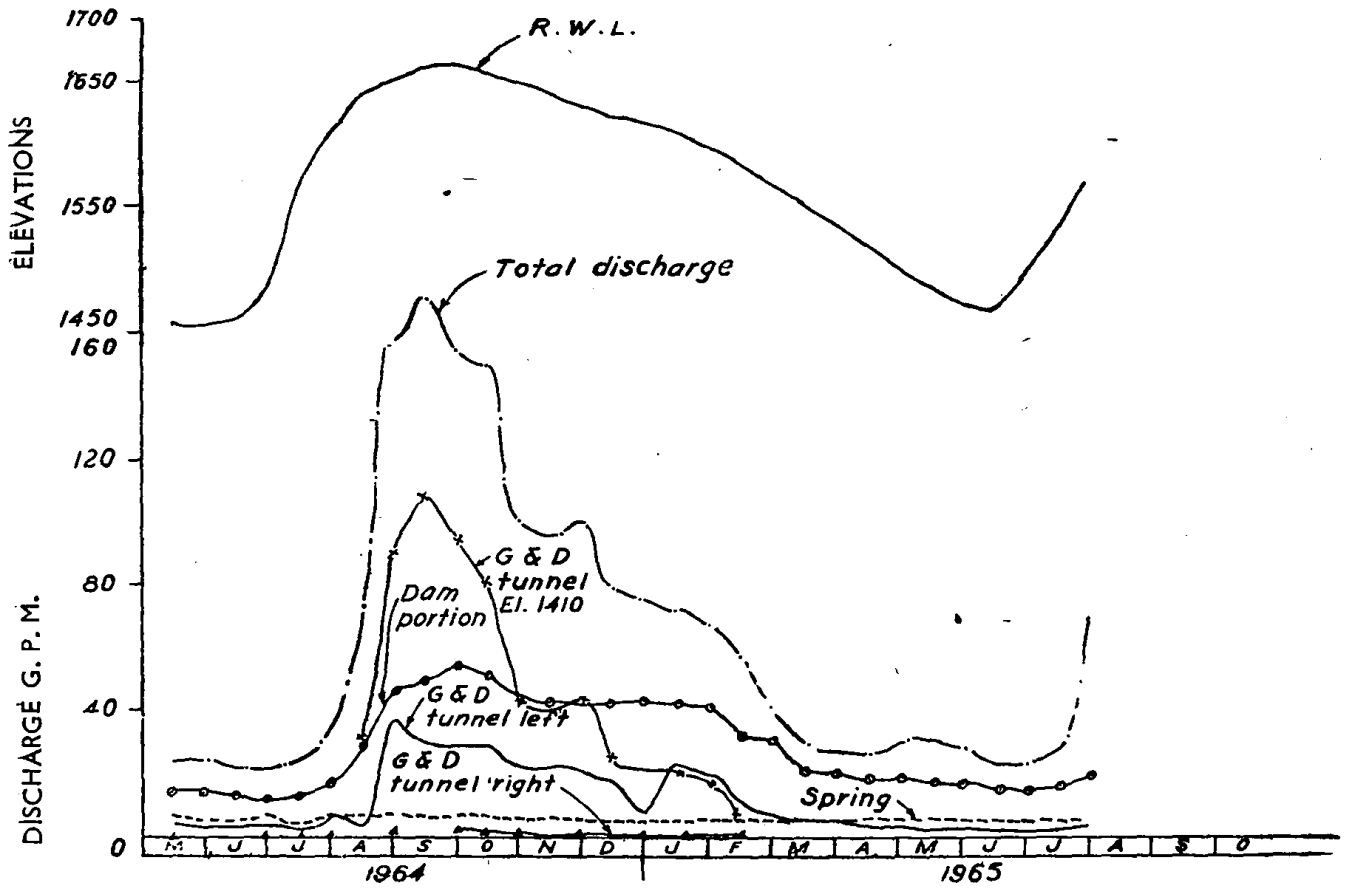


Fig. I-107



**REPORT
OF
THE COMMITTEE ON INSTRUMENTATION
FOR
DAMS & HYDRAULIC STRUCTURES**

PART II

DETAILED REPORT



सत्यमेव जयते

**MINISTRY OF IRRIGATION AND POWER
GOVERNMENT OF INDIA
NEW DELHI**

December, 1965

C. W. & P. C. No. 166

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(ii)

This report describes necessity for instrumentation of important structures, the objectives to be sought, the nature of the measurements to be made and a description of the various instruments generally adopted for such measurements.

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PART II
DETAILED REPORT

2-4 C.W. & P.C./66

1.0 OBJECTIVES OF INSTRUMENTATION

Dams and related works are built to last many decades, and form key structures to the development of river basin potential for irrigation and power generation. Inherent in the reservoir water storage are tremendous forces which the dam is expected to withstand throughout its operation life.

Consideration of the disastrous effect of the failure of a dam in terms of loss of life and property and the negation of planned benefits of the projects, make it imperative that means are available in the dam for providing information on continued assurance of its safety.

The objectives of instrumentation are two-fold. The instruments embedded in, or installed at the surface of, dams and other hydraulic structures keep a constant watch over their performance in service and indicate the distress spots which call for remedial measures. Thus these instruments play an important role in checking the safety of the structure.

In addition to serving the purpose of telling the "health" of the structure, the observation from the instruments form a cumulative record of the structural behaviour. The study of structural behaviour provides an important aid in modifying purely theoretical treatment so as to include the effect of actual field conditions. Most hydraulic structures are built on rather conservative assumptions to provide for the "unknowns" in the design. Observations from the instrumentation help reduce these "unknowns" and place the future designs on sounder footings.

The information obtained through measurements promotes the understanding of the influence of various parameters on the structural behaviour and

leads to formulation of more realistic design criteria. A few typical examples of the utility of observations are given below:—

- (a) Measurement of uplift at the base of concrete dams built by U.S.B.R. over long period of time led to development of the present U.S.B.R. "Uplift Criterion" applicable to concrete dams with a grout curtain and foundation drainage curtain downstream, whereby uplift head prevailing at the line of drains is assumed to be one-third of the net head.
- (b) Study of results of analysis of strainmeter data in Shasta Dam (USA) revealed the influence of columnar method of concrete dam construction on the distribution of vertical stresses establishing a non-linear distribution. This factor has now been included in the stress analyses procedure for computing stresses as corrected for construction programme.
- (c) Detailed study of the long time pore pressure measurements in rolled earth dams has led to the better understanding of the development of pore pressures and the factors influencing such development and formulation of more rational relationship amongst the various parameters involved. The present slope stability analysis procedures include this new concept for evolving more dependable and safer rolled fill high earthen embankments.
- (d) A study of temperature distribution in concrete dams, particularly concrete arch dams, has led to the identification of thermal load influence on the overall structural behaviour.

2.0 NATURE OF MEASUREMENTS AND THEIR PURPOSE

In order to achieve the objectives of instrumentation as described under para 1.0, it is of extreme importance that a systematic and complete plan of various observations/measurements must be prepared for obtaining as complete an information as is possible regarding the various internal or external, permanent or provisional loads, influencing the structural behaviour of dam and its foundations, and factors indispensable to interpretation of the results of such measurements. Measurements should be planned for the most important zones of the structure. Planning of measurement is made taking into consideration the results of the analytical and experimental (model) investigations, as also of the studies of the foundation and its specific problems, the investigation of which is considered of interest at the time. Various types of measurements made on previously built similar structure can aid in planning the necessary types of measurements for obtaining the needed information. It is essential that the various measurements made on the structure should be so planned as to provide information, not only of the individual components of structural action but also of the integrated structural performance of the structure and its foundation.

The planned measurements must provide for mutually checking information for establishing reliability of similar results, obtained from different methods of measurements, by comparative study of such results.

The nature of measurements and tests to be carried out on any structure will depend upon its dimensions namely height, thickness, span to height ratio, difficult foundation conditions.

However when working out a plan of observation, the cost of the equipment, of the observing personnel, and of the processing of observed data and interpretation of the results should be considered in relation to the cost of the dam and its situation. Limitation of cost of instrumentation in a structure may be a determining factor in deciding the types of measurements to be made on a given structure.

It is obvious that the methods of measurements, instruments, staff employed for observation and processing of observed data, and interpretation of results must be of a calibre needed to achieve useful results under the inevitable variability of conditions.

2.1 CONCRETE AND MASONRY DAMS

For observations of a concrete/masonry dam, a fully complete set of measurements as mentioned above is listed below. These measurements are grouped under three headings as under:—

- (a) Measurements indicative of structural behaviour *i.e.*, effects of loadings.

- (b) Measurements of external loading.
- (c) Measurements of properties *i.e.*, laboratory required for processing of observed data.

2.1.1 Measurements Indicative of Structural Behaviour *i.e.*, the Effects of Loading

2.1.1.1 Dam displacements in relation to points on ground (external)

- (i) Horizontal components.
- (ii) Vertical components.

The observation of the displacements of points on the structures, foundation and abutments, provides the simplest and sometimes sufficient means for providing positive information for ascertaining the overall structural behaviour of the dam and foundations under the imposed loading conditions.

As concrete and masonry dams, whether these are straight gravity or arch gravity, are constructed in blocks, separated by transverse joints, observations of displacements of the individual blocks under varying loading conditions reveal sudden and significant changes in displacements, resulting from local foundation discontinuities, and thus aid in determining the appropriate time for remedial measurements such as grouting of transverse joints, so as to ensure an integral and monolithic action of the structure and favourable spread of load on the foundation.

Trial load analysis of concrete dams is based on the overall deflections of the elements of the dam under specific loading conditions; measurements of deflections of the prototype structure would serve as a useful check on the analysis. No satisfactory comparison between theoretically computed and actually prevailing stresses in dam is possible unless actual displacements of the foundation, and their relationship with the mechanical properties of foundation rock as well as the influence of such displacements on the behaviour of the dams is detected.

Measurements made on the foundation of dams provide information to be used in control of displacement, inclination and distortion of structures. These are taken into account when installing equipment, determining the required structural safety, evaluating the working conditions of specific structural members and ascertaining the nature of settlement as time goes by.

Study of trends of foundation displacements under various loading conditions can aid in estimating the stability of the foundations, when there is consistency in the extent of deformations and the way deformations evolve and die out with time, and there are no sudden and significant discontinuities in the trends.

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Dependent upon the amount and type of related and supporting information available, displacement data can be used for determination of elastic and physical properties of concrete and its supporting foundations, particularly when the imposed loading has cyclic character.

The absolute displacement of a dam may be measured by Geodetic methods. The measurement of absolute horizontal displacements of points in the dam and its foundation in relation to points on the ground, is made by triangulation, by using precision theodolite, most commonly used being WILD T3. The vertical components of the absolute displacements are measured by precise levelling using WILD N3 precise level.

The telemetric measurement net work must necessarily rely upon stable datum points, not apt to be influenced by dam-reservoir loading, so that they can be considered as fixed for the purpose. By judiciously choosing these datum points, accurately measuring base line lengths and verifying the conservation of the scale and the orientation of the angle measuring device by appropriate means, precisions ranging from 0.5 to 1.0 mm can generally be attained in the determination of the absolute positions of the absolute marks and from 0.2 to 0.4 mm. in the measurement of their relative positions. To achieve such precisions, many precautions must be taken and extremely skilled operators are required.

The geodetic method has the advantage of measuring absolute displacements with a high degree of accuracy but it involves complicated calculations which require considerable time.

The delicateness of the triangulation system, the cost and the laborious calculations it involves prevent its being annually repeated a number of times that is sufficient and necessary to provide an accurate check on the behaviour of the dam. This is particularly true when the actions exerted on the structure change rapidly. Triangulation, therefore, should be confined to seasonal operations and supplemented by simpler and faster observations system. Moreover, triangulation should also be limited in space because grids of great extension cannot warrant an adequate precision, comparable with that given by other methods, since the errors spread very rapidly. Use of photogrammetry for the measurement of displacement surveys has been made but is not common practice on account of abundant difficulties in respect of equipment operation and subsequent analysis.

Another instrument used for absolute displacement measurement is Mekometer. The distance is measured by means of a modulated light-beam. The modulation frequency used at present ensures an accuracy of a few millimetres.

Displacements of points on top of dam may be measured by collimation method (Slide target method) but the end points have to be connected to an outside triangulation grid.

The absolute horizontal displacement of points in the interior of the dam can be measured by installing an inverted plumb line, which affords fastest, simplest, cheapest and most precise system, for such measurements. Vertical displacements of the foundation can be measured by deformation meters and also by settlement measuring devices.

2.1.1.2 Displacements of points in relation to other points in the Dam (Internal) i.e., relative displacements. (Horizontal Components)

Measurements of relative displacements of points in the interior of dam provide the fastest, simplest, most precise and direct method of watching the structural behaviour of the dam. Measurements of structural deformation over a considerable period of time (several years) furnishes information regarding the general elastic behaviour of the entire structure and foundation and provide a means for determining the elastic shape of the deflected structure which will permit distinction of load and thermal deflection components and with precise alignment data, provide for estimating the amount of translation or sliding.

Deflection cycles of remarkable uniformity in amplitude and period become clearly evident from the initiation of observations and any deviation is apparent, and study of these deviations help in detecting development of distress conditions in structure or foundation.

Dependent upon the amount and type of related and supporting information available, plumb line observations can be used for ascertaining the elastic and inelastic physical properties of the concrete and foundation rocks.

Relative displacements are measured by means of collimators and by the use of plumb lines with pendulums placed inside a shaft in the dam, measuring the displacements of the wire normal to and parallel to dam axis (radial and tangential direction in case of arch dams) in straight gravity dams, with respect to fixed points in the shaft.

Though the measurement of displacement by pendulums is restricted to significant individual point or change of direction of significant lines, pendulums are the foremost instruments for the observation of behaviour of dam.

Using improved optical coordinometers, micrometer microscope, accuracy as high as two thousandth of an inch can be obtained.

2.1.1.3 Measurement of Tilt (Rotation)

The measurement of tilt (rotation) is made to determine displacements. It is, however, an indirect method as to obtain displacements, it is necessary to integrate the rotation diagrams.

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The rotations usually measured are those taking place in vertical planes as these can be easily made with air-bubble clinometers, inclinometers (tiltmeters).

The rotations about vertical axis can only be determined indirectly through the measurement of horizontal displacements. The clinometers are mounted on suitably designed bases, fixed in the gallery walls in the longitudinal and transverse galleries provided in the dam.

2.1.1.4 Measurement of Stresses

Direct measurement of stresses in dams is of the greatest importance, from the view-point of watching the structural behaviour of dams and foundations, verification of the stresses obtained by experimental and analytical methods, and for establishing the actual stress distribution patterns, resulting from imposed loads. Study of deviations in actual stress distribution patterns from the theoretically derived ones aid in distinguishing the influence of factors and phenomenon not considered in the analytical and experimental investigation and such findings can be advantageously used for improved and economic design in future.

Inconsistencies or discontinuities or sudden and significant changes in the continuous stress plots indicate developing foundation deformations, valley adjustments or distress condition in the structure and when studies in conjunction with continuous record of measurements of the instruments in dam, provide appropriate advance signal for ensuring timely remedial measures, if warranted, to arrest distress trends from attaining adverse proportions.

Stresses are measured by embedding stressmeters of the Carlson type or the vibrating wire type.

The stress-meters afford the advantage of direct measurement of stresses, eliminating the involved and time consuming processing of data recorded by strainmeters, requiring knowledge of the mechanical properties of concrete, such properties, beside being complex, generally show important variations from point to point and also change with temperature and other conditions of concrete. But it suffers from the disadvantage that it can measure only compressive stresses and not tensile stresses.

Not only special care has to be taken for installation of stress-meters but these are also very costly.

On account of the heterogeneity of the stone masonry direct measurement of stresses by use of stressmeters cannot be made.

2.1.1.5 Measurement of Strains

Measurement of strain in concrete is made by embedding strainmeters.

As most of the design methods are based on limiting the stresses, the observation of real stresses developed in structures is essential for the assessment of the structural behaviour, improvement in design methods or to the control of safety.

As the device developed and used to record stresses directly is not capable of recording tensile stresses, besides being very expensive and requiring much more care for installation than that required for strainmeters, in majority of cases, it is necessary to determine stresses from strains. For this reason the techniques for measuring strains are very important.

The apparatus must be delicate, possess stable characteristics for long-term observations, since in dams there are loadings which develop slowly, such as the weight of dam itself, the hydrostatic pressure on the dam, the settlement of foundations, temperature variations in a thick element, shrinkage of concrete.

All strain measurements in concrete are made with apparatus placed inside the concrete as measurement of deformations in points far from the surface has the advantage of avoiding disturbances due to the surface cracking of concrete. When measurements in the interior are made, special attention must be paid to the shape and rigidity of the strain gauge so as not to upset the strain to be measured.

To meet the requirements of limiting the length of the strain gauge and giving it a high deformability in order to guarantee that the gauge will follow the deformation faithfully, without the necessity of large stresses developing between the strain gauge and material in which it is embedded, strain gauges with ratio between length and diameter varying between 5 to 10 are usually used for observations of strains in concrete.

Depending upon whether two-dimensional or three-dimensional fields of stress are required to be observed, strainmeters rosettes for recording two-dimensional or three-dimensional strains, consisting of 6 and 9 meters respectively, are embedded in concrete.

Duplicate meters are provided along three principal axes to improve the reliability of results and to allow determination of stress field even when some meters become inoperative or erratic.

"No stress" strainmeters are installed for measurement of concrete deformations due to autogenous growth, moisture content, shrinkage etc. for effecting necessary corrections in the observed deformations, with a view to obtaining strains, as resulting from stresses only. Measurement of strains near the upstream and downstream faces of the dam are made for determining the strain gradients near the surfaces of structures. These measurements aid in determination of elastic properties of the *in situ* concrete.

Strain measurements made by installing strainmeters at varying depths into the foundation under the

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dam provide information on the foundation deformation behaviour and data can be used for assessing the elastic and in-elastic properties of foundation rock.

Continuous record of strains, in dam concrete and foundation can give direct indications of structural adjustment in foundation and dam, and distress condition developing, by the presence of discontinuities and sharp and sudden changes in the time plots of strains.

Measurement of strains in the penstock liners and reinforcement around penstock openings or other openings in dam, provide information on the progressive development of stresses in these parts and magnitude of stresses actually prevailing under operating loads. This information can aid in improvement in the design of penstock liners and reinforcement around such lined openings.

2.1.1.6 Uplift in the Body of Dam (Pore Pressures)

Stress and stability analysis of masonry dam (concrete or stone masonry) is carried out by considering the existence of uplift across every horizontal plane, having uplift intensity distribution in accordance with the design criterion in practice. The effect of the uplift is to induce instability on account of the resulting buoyancy in weight of material in dam, above the horizontal section under consideration. However, concrete even though not completely impervious, has a very low permeability and as such pore pressure development in concrete mass subsequent to reservoir filling would be very slow. In the case of stone masonry dams the permeability will depend upon heterogeneity of the masonry and accordingly pore pressure development may be at slower or faster rate, even though the operating heads in the case of stone masonry dams would be much smaller compared with those to which the high concrete dams are subjected.

Provision of arrays of electric resistance type or vibrating wire type pore pressure cells in concrete/masonry at different elevations, and spaced at suitable distances from the upstream face, would provide information on the rate of penetration of seepage (permeability) into the concrete/masonry and on the development of pore pressure.

Since large concrete dams are provided with row of internal formed drains located a few feet downstream from the upstream face, record of the pore pressure development and its variations would indicate the effectiveness and adequacy of these drains. At the same time, any sudden and significant variations in the pore pressures development may be indicative of some structural damage or deficiency in the dam material, such as crack formation or porous concrete respectively, warranting timely remedial measures being undertaken.

On account of the slow penetration of the reservoir water in the concrete pores the stability of the Carlson

type meters for long-term observation has not yet been ascertained.

2.1.1.7 Temperatures of the Dam Interior

In concrete dams, particularly the massive ones, volume changes, resulting from temperature changes, have been considered responsible for most of the cracking and thermal stresses in concrete dams. These temperature changes result from the heat of hydration of cement in concrete and the annual seasonal changes radiant heat etc.

To minimise this cracking, large straight gravity and arch gravity concrete dams are made in blocks separated from each other by transverse and longitudinal contraction-joints and cooling of mass concrete is resorted to, to enable each monolith to attain its minimum volume, when stable temperature conditions are attained. To restore the dam to its monolithic state for integrated behaviour, contraction-joints are grouted with cement grout, when maximum joint openings become available, resulting from mass concrete cooling operations.

Thin arch dams, however, have radial transverse joints but no circumferential joints, like those in arch gravity dam, but temperature load is most significant in case of thin arch dams.

The principal purpose of the temperature measurements in concrete dams, is to aid in control of cooling operations, and construction programme and in conjunction with the joint movement measurements, to choose the best time for grouting the contraction joints.

Subsequent to the completion of contraction joint grouting, measurements of temperatures would provide the necessary information for establishing the thermal stability of the structure as influenced by the air and radiation temperatures on the downstream face and the reservoir water temperatures on the upstream face.

As temperature variations are among the important loadings on concrete structures from the point of view of displacements, joint openings, strains, and stresses, accurate measurement of temperature at various points in the dam is indispensable for interpretation of the results of these measurements. Measurements of temperature in concrete around the openings in dam for conveyance of reservoir water, would aid in establishing the zone of temperature gradient. This information can be used to check the assumption of temperature change adopted in the design of reinforcement around opening and may lead to economical design of reinforcement around openings.

Inconsistencies in the record of temperature measurements would be indicative of some unusual occurrence in the structure and necessitating initiation of appropriate measures for control of the same.

Temperature measurements may be made by embedding resistance thermometers (Carlson) or thermocouples or vibrating wire type (Maihak or Telemac) thermometers and thermistors.

2.1.1.8 Joint Movements

Measurements of joint movements are made to determine, in conjunction with the temperature measurements during cooling of mass concrete the best moment for grouting of the contraction joints. As mentioned in para 2.1.1.7 grouting of joints is done, when the joints have opened to their maximum, as grouting done at any other time, can endanger the safety of the dam on account of resulting impaired monolithic action and structural integrity.

Subsequent to the completion of the grouting of contraction joints, measurements of joint movements, provide information regarding the behaviour of the grouted joint and in interpretation of the results of observations of structural behaviour of the dam.

Measurement of joint movement inside the dam is made by joint-meters of resistance type (Carlson) or vibrating wire type (Maihak or Telemac) and those at surface by strain gauges. Sliding of the joints at the faces is measured by detachable gauges like Whittemore and Marion type. Progress in cracks can be assessed by crack detectors which are simply long-gauge joint meters.

2.1.1.9 Seepage (Infiltration)

In order to reduce seepage and uplift pressure under the dam foundations, grout curtain is provided near the u/s face for a considerable depth below the base of dam. This provides a comparatively watertight barrier to percolation from reservoir. A few feet d/s of this grout curtain, is provided a drainage curtain, by drilling a line of holes from the foundation gallery and tunnels in the abutments, to intercept any seepage that passes d/s of the grout curtain. Measurement of seepage and uplift d/s of the grout curtain provide a direct indication of the adequacy and effectiveness of the grout curtain, drainage curtain and functioning of the drains, and reveal when and where remedial measures may be required. Observations of leakage from contraction joints (ungROUTED), lift joints and cracks provide a means for judging the quality of workmanship as well as disclose the necessity for corrective measures to preserve the integrity of the structure. Study of seepage and uplift also provide information about the over-all state of the grouting, and sudden changes in the recorded trends indicate need for remedial measures in specific areas. Seepage measurements made at various locations in the foundations and abutment may indicate need for increased drainage facilities at some other safe location to relieve the dam areas of uplift, not caused by the reservoir, but by underground water. Results of chemical analysis of samples collected from drain holes provide information as to whether or not there is foundation material wash in the effluent. Presence of

material wash would indicate need for appropriate remedial measures in the specific areas. Study of quantum of seepage, under varying reservoir level, with the expected seepage, aids in assessing the influence of the geological formations in the dam and reservoir foundation on seepage development.

2.1.2 Measurement of External Loadings

The measurement of external loadings are imperative to define the characteristics of the various kinds of loading experienced by structure during periods of construction, concrete coolings, joint grouting, reservoir filling and subsequent service operation, to enable realistic interpretation of results of structural behaviour, obtained by processing data, and assessment of the response of the structure, and to establish relationship of the variation in loadings with the effects produced in the structure by such loadings, to define limits of the normal and safe behaviour of the prototype structure, and provide basic guidance for watching the structural behaviour and safety of the structure under imposed loading conditions during its operative life.

Measurement in respect of the following items should be made:—

2.1.2.1 Hydrostatic Pressure

Measurement of the hydrostatic pressure resulting from operating headwater levels and tailwater levels should be made by installing automatic continuous water level recorders in shafts, suitably located in the dam. Where power plants are built at the toe of the dam or near it, automatic continuous level recording equipment for record of tail-water level may be installed in shaft suitably provided in the power plant building and connected with the tail-water. In cases, the provision of automatic level recording equipment is not contemplated, daily headwater and tail-water, levels should be observed by means of gauges (scale) painted on the upstream and downstream face of the dam, at locations conveniently visible.

Measurement of the hydrostatic pressures, being one of the two principal loading to which structure is subjected, is essential for interpretation and realistic assessment of the structural behaviour of the reservoir retaining structure, as obtained from various measurements recorded by measuring devices and instructions provided in the structure for this purpose.

2.1.2.2 Uplift

Uplift is one of the most significant instabilising force in concrete and masonry gravity dams, and measurement of uplift in the foundation provide direct indication of (i) the prevailing magnitude of uplift resulting from operating reservoir heads; (ii) effectiveness of the grout curtain provided by drilling and grouting deep holes into the foundation close to the upstream face, for providing a water-tight barrier to seepage from reservoir and reducing uplift;

(iii) efficiency of the drainage curtain provided by drilling a line of vertical holes to specified designed depth, a little distance downstream of the grout curtain, for interception of the seepage that penetrates the grout curtain; and (iv) the influence of the geological features in the foundation on the development of uplift. From study of the uplift record, it can be ascertained, whether the uplift values are within the design criterion values and in case these exceed the criterion value, the need for undertaking suitable remedial measures at specific areas of excessive uplift.

Measurement of uplift pressure in the foundations of concrete and masonry dams is made by means of uplift pipes installed in concrete/masonry and led into nearby galleries where their terminations are fitted with Bourdon type pressure gauges for record of uplift as it progressively develops. The pipes are Commissioned into functioning by drilling through these pipes, holes 3 ft. deep into the foundation. Electrical transducers are also used for uplift measurements.

2.1.2.3 Measurement of Reservoir and Air Temperature

Measurement of reservoir water temperature at varying depths can be measured by installing resistance thermometers or thermo-couples flush with the upstream face at specified elevations.

Measurement of maximum and minimum air temperature should be made at the site of the structure, preferably near the dam. A small meteorological post should be established at the dam site for such and other meteorological measurements.

Measurement of temperatures of reservoir water and air is essential for interpretation of the results of structural behaviour, understanding of the thermal condition of the structure and for distinguishing the effect of temperature load in such measurements as deflections, settlements, stresses and strains and joint movements.

Influence of solar radiation on the final thermal stability of the structure can be determined only if the above measurements are available in addition to the dam interior temperatures.

2.1.2.4 Rain

Measurement of rainfall locally should be made by installing rain gauges in the vicinity of the site, as this measurement is necessary for interpretation of the uplift and seepage development. Record of rainfall in catchment for estimating floods is excluded.

2.1.2.5 Ice Effects

This measurement needs to be made in regions where reservoir waters are subjected to freezing in winter. Information is to be used for interpretation of the results of structural behaviour stresses and deflections etc.

2.1.2.6 Dynamic Loads—Earthquakes

Measurement of vibration in dams located in areas, subjected to seismicity, should be made by installing seismographs and accelerometers at appropriate locations in or near the dam, which are free from extraneous vibrations, so as not to vitiate the actual records of vibration due to seismic shocks or the foundation disturbances resulting from dam and reservoir loading. The equipment should provide information for determining the natural period of vibrations of the dam under the impressed shocks. These measurements will provide information regarding the dynamic response of the structure under disturbances experienced by it, and can be used for checking the adequacy of assumptions, made for a seismic design of the structure, and actually resulting factors of safety under dynamic loads and thus aid in improvement of the criterion for the earthquake resistant design of the structures.

2.1.3 Measurement of Properties in Laboratory required for processing of Observed Data

The many properties of concrete of the structure being tested must be defined exactly as they are, and as they vary with time. Every effort must be made to duplicate as closely as possible the exact materials and the exact combinations of these materials in the structures under test.

The properties that must be defined are—

- (a) Elastic and inelastic properties namely:—
 - (i) Modulus of elasticity at various ages.
 - (ii) Poisson's ratio.
 - (iii) Creep properties at various ages.
- (b) Thermal properties which include—
 - Coefficient of thermal expansion.
- (c) Autogenous growth.

Other properties which are of interest but do not enter directly into the problem of stress determination, are heat of hydration, thermal diffusivity, unit weight, specific heat, conductivity and strength properties. The above five factors are important in reducing strain observations to stress. The other properties are primarily useful in predicting temperature effects and in artificial cooling operation.

2.1.3.1 Elastic and Inelastic Properties

- (i) Modulus of elasticity at various ages
- (ii) Poisson's ratio

Of these two properties, the modulus of elasticity is more important and should be measured without fail on every project where strain-measurements are made. Poisson's ratio can be estimated closely enough. Both of the properties should be measured on the same specimen of the material by mounting dial

gauges on three gauge lines for measurement of elastic modulus and one gauge to measure diametral changes for Poisson's ratio measurements and loading the specimen in accordance with standard laboratory procedures.

Measurement of the modulus of elasticity and Poisson's ratio can be made by embedding one vertical strain-meter located at the centre and two horizontal strain-meters mutually at right angles at the centre of standard specimen for recording axial strain and lateral strain. This test should be undertaken on the same specimen at various ages. Determination of Poisson's ratio is essential for computations of unidirectional strains from strains recorded by group of strain-meters in the direction of strain-meter alignments. Measurement of modulus of elasticity is required for conversion of strains into stress.

(iii) *Creep at various ages*

This test requires more than the usual variety of laboratory testing machines as the load must be sustained without appreciable change for years at a time. Creep is a property of concrete that causes it to continue to deform for long periods after it has been loaded. It varies with the age of concrete at which it is first loaded and also with the duration of the loading. Determination of creep properties should be made on specimens which are large enough to permit closest resemblance of the prototype concrete with the concrete being tested.

To avoid failure at upper limit of loading, the specimens should be loaded at one-third the ultimate compressive strength, appropriate to the age of loading and may vary from 75 psi at 2 days loading to 800 psi at one year loading.

Two specimens should be made for initial loading at ages 2, 7, 28 and 90 days and 1 year. Additional pair of control-meters is necessary for check on weight loss and autogenous movements.

Determination of creep characteristics of material is required for eliminating the creep strains from computed strains for correct stress evaluations from strainmeter data.

2.1.3.2 *Thermal Properties*

(i) *Co-efficient of thermal expansion*

For this test a pair of sealed 8×8×48 inch specimens should be used, filled with concrete, exactly like that in the dam, except for omission of all aggregate over 3 inches in size. The specimen should be subjected to temperature cycles of 40°, 70° and 100° and back to 70° at various ages preferably 2, 7, 28 and 90 days and 1, 2 and 5 years. Length changes can be measured by external strains gauges on inserts cast into the specimen concrete, or internally using a Carlson strain-meter. Measurement of co-efficient of thermal expansion is essential for applying temperature corrections for obtaining strains from strainmeter data and also in analysis of deflection due to temperature changes.

2.1.3.3 *Autogenous Growth*

Two sealed specimens similar to the co-efficient of expansion specimens (ref. to para 2.1.3.2) are simply stored at standard 70°F temperature, and measured periodically to see whether there are any length changes under condition of no temperature change and no moisture change. The latter condition must be checked periodically by weighing accurately.

The measurement is made to ascertain the growth characteristics of the material, if present for use in excluding the length change effect due to growth, from the actual length changes, recorded by strain-meters for computation of strains.

2.2 EARTH DAMS

Earth dams differ from masonry and concrete dams because of the relatively greater deformability and higher permeability of earth masses (excluding plastic clay hearting). Strains and displacements in earth dams are therefore very much larger, hence comparatively simple instruments can be used for measurements of strains and displacements. On the other hand due to the large cross sectional areas greater heterogeneity of earth, larger number of instruments have to be installed in earth dams. Distribution of stress in earth dams is more complex and the design analysis is based on radical simplification of the stress pattern and shape of rupture planes. Consequently stress measurements require considerable judgment in interpretation. Seepage is of far greater significance as it can cause internal erosion as well as increase in pore pressure resulting in instability.

2.2.1 Measurement of Displacements

2.2.1A. *Internal Displacements*

Internal measurement of displacements provides useful indication about the behaviour of earth dam. Vertical movements indirectly indicate the stresses while horizontal movements would reveal dangerous incipient sliding or shear failure. Equipment for measuring vertical movements consist of sections of steel tubing with cross arms which ensure that each individual section moves along with the adjoining earth-mass (see figure 29-2 USBR Earth Manual, designation E-29). Sections of the pipes are isolated and movements of each section are measured by a simple sounding device with reference to the top of the installation. Elevation of the top of the installation is measured by accurate levelling with reference to a datum. Internal horizontal movements are measured by two alternative methods. In the first method, inclination of a plastic tubing is measured by a sensitive electrical inclinometer, step by step, starting from the bottom of the tubing. As the tube is anchored at the bottom into a firm unyielding stratum, horizontal displacements are computed by integrating the movements of each section of the tubing on the basis of changes in the inclination. Potential sliding surfaces can be located by observing areas where abrupt changes in

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the inclination take place. Instruments for reading internal horizontal movements have also been developed on principles similar to the vertical cross arms. In these instruments, the relative movements between horizontal cross arms are measured by transmitting the same by means of cable to a pair of counter weights (see figure 30-1 USBR Earth Manual, designation E-30). In problems of long-term stability measurement of internal horizontal movements facilitate identification of potential slips or failure surfaces. As methods of estimation of long-term (Creep) strength are still in the stage of development, instruments for measurements of internal movement are obligatory when such problems arise. Installation of internal movement measuring instruments is evidently essential for test embankments in order to locate the failure surface.

2.2.1B. Measurement of Surface Displacements

In earth dams, ordinary surveying methods are adequate for measuring the vertical and horizontal movements of the surface. In spite of their simplicity, with reference to measurement, these are very useful as they can reveal potential bulging or abnormal yielding leading to slide or shear failure.

2.2.2 Measurement of Stresses

2.2.2A. Measurement of internal total stresses in earth dam is complicated because the stress distribution is influenced by the differences in the stress strain characteristics of the instrument and those of the surrounding soil mass in shear as well as in compression. Soils do not possess unique stress strain relations; strains are liable to be radically influenced by plastic flow hence stress strains relationship reveal marked hysteresis. Therefore it seems to be practically impossible to fabricate a stress-meter which will have identical deformations as the soil mass. Although a number of indigenous devices have been developed for measuring earth pressures, no reliable method is developed so far, for estimating the total stresses within an earth mass. Nevertheless earth pressure cells have been installed in Europe and U.S.A. in some of the large earth dams during the last few years. These may however provide useful qualitative indications of the variations in the internal stresses with changes in the loading condition, i.e., the embankment height, reservoir pressure. Current practice is based on limiting values of principal stress & most unfavourable ratios of principal stresses; hence stress measurements within the earth mass, prior to failure, cannot be used directly for verification of design assumptions. Measurements of stresses in earth dams would, therefore, be required only in a few cases for special research, connected with the analysis of the internal stresses in earth dams. Equipment for measuring internal stresses in earth masses is essentially similar to the cells used for measuring contact pressures of earth and rigid surfaces such as concrete or masonry, except that the cell is provided with diaphragm on both the faces.

2.2.2B. Reliable measurement of contact pressures, where earth is placed against masonry or concrete structures, is on the other hand, quite practicable as long as the pressure cell deformation agrees closely with that of the wall or structure. This is generally possible as the material of the wall or structure possesses well defined stress strain characteristics. The reliability of measurement in such cases depends on the size of the area of contact (in order to average out minor variations in the compaction of the soil and the proportion of rock and coarse grained material) and the selection of adequate number of locations to ensure representative coverage of the factors influencing the earth pressure distribution.

2.2.3 Measurement of Strains

Internal measurement for vertical and horizontal displacements by means of cross arms, provide adequate information regarding strains in earth masses. The gauge lengths (5 to 10 ft.) involved are large enough to average out local non-uniformity. At the same time, they are small enough as compared to the large cross section of earth dams. The relevant movements of cross arms, vertical or horizontal, directly indicate the strain in the earth masses.

2.2.4 Measurement of Pore Pressure

In the design of earth dams the greatest uncertainty lies in the estimation of pore pressures and analysis is generally based on simplifying assumptions regarding the magnitudes & ratios of principal stresses and drainage (or dissipation). It is difficult to estimate the average compressibility and average water content of the embankment & foundation material because of unavoidable heterogeneity and unknown anisotropy. Pore pressure measurements, therefore, assume the first importance in instrumentation of earth dams.

As prediction of pore pressures depends on compressibility of soil, measurements of internal compression and settlement must supplement pore pressure measurements. Internal compression and settlement records also provide indirect indications of the effective stresses *in situ*.

Measurement of pore pressures is also essential for evaluating the performance of measures in control of seepage through or under the dam. Failure of drainage provisions and dangerous concentration of seepage can be revealed by piezometric observations. Piezometers for evaluation of seepage control measures generally consist of simple stand-pipes as percolation through pervious material is involved. On the other hand, piezometers in less permeable strata consist of close system comprising a measuring tip connected to the pore gauge through single or double tubing. Electrical devices are also used for measuring pore pressures.

They register the changes in pore pressure either by equalising pressure against a diaphragm or by recording movement of the diaphragm by electrical strain gauges.

Measurement of seepage and corresponding levels of the reservoir and tail water is essential for interpretation of pore pressure observations.

2.3 EARTH RETAINING STRUCTURES

Sub-para (b) of Para 2.2.2 applicable.

2.4 TUNNELS AND ROCK STRUCTURES

Tunnels and other rock structures such as foundations and underground caverns present major design and construction problems in hydroelectric and irrigation projects. Tunnels and other rock structures also account for a substantial amount of the total cost of the structure. However, the design of these structures is highly empirical. Systematic instrumentation on these structures has not only resulted in a better understanding of the structural action but also in a substantial reduction in the cost.

2.4.1 Data for Design

The design of the tunnel linings depends on the type of tunnel whether free or pressurised and the characteristics of the rock. *In situ* measurements are necessary prior to design for determining the elastic modulus and Poisson's ratio, residual stress, and vertical and horizontal earth pressures. Various theories are then used to determine the sharing of the loads between the lining and the rock. Hydrostatic pressures built up behind the steel lining may lead to buckling and become a critical factor in the design. Reliable field data on these pressures may lead to considerable reduction in the lining thickness. There are other important problems such as dynamic stresses due to adjacent blasting and stresses around interconnections which need detailed field data to improve on the current practice in design. Some of these factors are discussed below with emphasis on the type of instrumentation necessary.

2.4.2 Residual or Locked-in Stress in the Rock

An assessment of the residual stress in rock is of vital importance in the economic design of the lining for underground pressure tunnels. If sufficient precompression exists around the tunnel the lining may be entirely eliminated or a thin lining used. The prestress in the rock increases its capacity to carry internal pressure. The greater the precompression the greater the load that could be carried without developing tensile stress.

There are two main techniques used in the measurement of residual stresses in rocks. The first depends on the measurement of the change of strain on the surface of the tunnel on isolating a small area around it by drilling or other means. The strain is measured by SR-4 type of gauges. The other is the flat jack method in which a slot is made in the rock and the displacement is measured by dial gauges. The initial zero reading is then brought by applying pressure through the flat jack.

2.4.3 Vertical and Lateral Rock Pressures

Estimates of vertical and lateral pressures are of great importance in the design of the tunnel supports and their spacing during construction and in the design of the lining later. Realistic estimates based on field measurements can lead to great economy measurements in the Garrison Dam test tunnel were designed to give this type of information. Measurements were made by invar tape, dial gauges and electrical resistance gauges.

2.4.4 Sharing of the Load between the Lining and Rock

Considerable saving in steel can be effected whenever it is possible to bury penstocks in solid rock and the latter is allowed to share part of the hydrostatic load. Although the rock modulus is estimated by various methods such as the testing of cores, measurement of the ultrasonic wave velocity, these tests do not give the correct value to be used in the design directly. Data obtained from the *in situ* tests using various test sections which are pressurised and instrumented have proved most reliable. The measurements of the changes in the diameters and the strains are made by electrical resistance strain gauges. Some of the most elaborate tests of this type were made for the design of the penstocks of Kimono hydroelectric power plant for the Aluminium Company of Canada. There a ten feet diameter sphere was embedded, and pressure upto 3600 psi was applied. Strains were measured in the surrounding rock by Carlson strain meters.

2.4.5 Buckling Pressures

Various theories have been propounded for the analysis of penstocks under external hydrostatic pressures. Field observations are of great significance in arriving at the minimum thickness of steel linings. Measurements made at Koyana by Carlson strain gauges in the concrete and steel lining has given useful data for future designs.

2.4.6 Interconnections and Dynamic Loads

Stresses around tunnel interconnections in the rock and the lining are difficult to analyse and field data has to be collected to rationalise the designs. Procedure and instrumentation similar to that cited under para 2.4.4 could be adopted.

Another important factor to be considered during the construction of tunnels is the effect of adjacent blasting on the completed tunnels. Instrumentation is very helpful in such cases in arriving at a safe charge.

2.4.7 Other Rock Structures

Other rock structures of importance in river valley projects are underground caverns for power houses and other control structures and the foundations. In

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addition to the problems presented by tunnels in general these structures present some special problem. Upheaval and relative ground movement are important in the underground power houses as these may affect the alignment of the generator shafts.

Rock foundations with weak seams and fault zones below dams also present serious difficulties and field measurement of the relative settlements and movements across the contact zones could suggest some improvements in the future designs.

2.5 REINFORCED CONCRETE STRUCTURES

Reinforced concrete structures concerned with river valley projects alone are discussed. These include aqueducts, bridges, power house foundation structures, intake and control structures, spillway piers, conduits, galleries, shafts and sluiceway and sump chambers of other openings in dams. Most of these structures are highly indeterminate and three dimensional. Although two dimensional analysis is frequently made very little factual data on the validity of such simplifications has so far been obtained. The action of reinforcement in such complicated structures as scroll cases in power house structures or the generator floors is hardly understood and analysis as simple beams and columns leads to a very uneconomical use of steel. Measurement of the actual stresses, strain and deflection alone can decide the issues. Measurements are being made now in most advanced countries on R. C. C. structure to improve designs. The general nature of problem is discussed with respect to each type of structure below.

2.5.1 Aqueducts and Bridges

Large aqueducts and bridges present difficulties in design due to continuity between the floor and the beams, bow string arches and the tie and stresses in the hangers. The design calculations are complicated and knowledge in this field could only be extended by actual measurements.

In prestressed concrete structures such as bridges and syphon aqueducts measurements are necessary to check the construction and generally are stipulated in contract. Measurement of leakage, stresses in concrete and reinforcement and deflection are required to verify the calculated values.

2.5.2 Power House Structures

The heavy reinforcement required both for the foundation concrete enclosing the scroll case and in the generator floor in the absence of a satisfactory and rational method of design emphasises the urgent need of instrumentation in these structures. Measurements of stresses and strain on the concrete surface and interior points and in the reinforcement bars are required.

These necessitate the use of electrical Carlson type gauges for concrete reinforcement and SR-4 type surface gauges for concrete. The deflection could be measured by dial gauges.

The measurement of the vibration characteristics of the floor and its response to the imposed vibration of the machinery or blasting requires the use of acceleration and velocity pick ups.

2.5.3 Openings such as Galleries, Shafts & Sluiceway in Dams

There is large number of openings in modern dams and these are heavily reinforced. Even in masonry dams the portion around these openings are constructed of reinforced concrete. The reinforcement around openings is generally based on elastic theory but there are divergent opinions regarding the reinforcements. Only field observation of the stresses and strain can give this information. The effect of external load, shrinkage and temperature and internal pressure is to be studied. The redistribution of stresses by the cracks and plastic yielding could, if confirmed in the fields lead to a reduction of the reinforcement required and to large economies.

2.5.4 Conduits

Conduits form important structures in many of the earth dams. The entire economics of the conduit design depends on the distribution and magnitude of the horizontal and vertical earth pressures. The measurement of these pressures in conduits under construction and the measurement around few selected points may lead to improvements in future designs.

2.5.5 Spillway Piers and other Control Structures

The importance of a detailed knowledge of the stresses in the spillway piers lies on the fact that every reduction in the width of piers and increase in height leads to large economy by providing greater spillway length. This, however, should not affect the safety of the structures. Measurements are necessary of the stresses and strain in the concrete and reinforcement as well as of load in the gate members such as the end arms. The effect of both static and dynamic loads is to be studied.

Similar problems are presented by regulator for canals in outlet tunnels.

2.5.6 Towers and Framed Structures

Intake towers for hydroelectric projects are subjected to wind load, wave action and temperature differentials apart from the loads due to self weight and other mechanical parts such as trash racks and gates. Earthquake loads may be important in some areas. The assessment of the loads acting on these structures and the complex response of the structure has to be based on factual data to arrive at economical designs. The measurements of loads, temperatures, stresses and strains in concrete and reinforcement and the total deflections are of importance in improving these designs.

3.0 INSTRUMENTS

Various types of measurements required to be made for assessment of the structural behaviour of dams and other hydraulic structures have been described in the preceding chapter 2.0. In this chapter are described the different types of instruments, measuring apparatus and devices which are generally used for making these measurements. These instruments or apparatus may be either of the embedded type or the surface type. The former type of instruments are required to be embedded in the body of the structure, whereas the latter type are meant to be provided on the exposed and accessible surface of the structure. Based on the principle of working, the instruments have been classified in the following general three types:

1. Optical, 2. Mechanical, 3. Electrical.

Instruments described in the following pages relate to the following types of measurements :

1. Displacements.
2. Stress.
3. Strain.
4. Pore Pressures.
5. Temperature.
6. Applied (External) loading.

For each of these measurements, different types of instruments in use have been described. While sufficiently detailed description has been given for instruments generally adopted for measurements, indicative of structural behaviour, only brief reference has been made to such of them, as find mention in relevant literature and are in the process of development and testing.

For each type of such instruments, detailed information in respect of the following five items has been given :

- (i) Principle of working & construction details.
- (ii) Number, location and layout.
- (iii) Method of installation.
- (iv) Method of observations, forms of record, sources of error etc., etc.
- (v) Method of analysis of observed data.

3.1 INSTRUMENTS FOR MEASUREMENTS OF DISPLACEMENT

Instruments for measurements of displacement described in the following pages are grouped as under :

- (a) Those required for measurement of displacements in concrete and masonry dams.

- (b) Those required for measurement of displacements in earth dams.

- (c) Those required for measurements of displacements in other structures.

3.1.1 Concrete and Masonry Dams

The following instruments for making the measurement of displacements of the type mentioned against each, are covered in the following paras :

1. Optical instruments for measuring displacement by geodetic methods.
2. Plumb lines for measurement of relative dam deflections.
3. Collimators for measuring displacement by collimation method (Alignment survey method).
4. Detachable gauges for surface displacement measurement.
5. Settlement measuring device for foundation displacement measurement.
6. Reverse (inverted) plumb lines for measurement of dam and foundation horizontal displacement.
7. Joint meters for measuring internal joint movement.
8. Deformation meters for measurement of foundation deformation (vertical).
9. Tiltmeter (Clinometers) for measurement of tilt (rotation).

3.1.1.1 Measurement of Displacement by Geodetic Methods

A. Geodetic methods used for measurement of deformation/displacement of concrete/masonry dams essentially involve the use of optical instruments and are thus classified as optical methods.

Measurement of deformations/displacement of concrete dam and its foundation is made by measurement of the absolute horizontal displacement of points (targets) located on the downstream face of the dam and abutment in relation to points on the ground by precise triangulation. Wild T3 type theodolite has been commonly used for such precise work.

The vertical components of the absolute displacements are made by precise levelling by using precise level. Wild N3 level has been considered to be most suitable for such levelling. Periodic observations of each target are made using first order theodolite from pairs of piers (instrument stations) suitably located

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downstream of the dam from considerations of the intervisibility of the instrument stations and target, and base line stations and well conditioning of the triangle. To provide a stable base to the telemetric measurement net work, the instrument stations in the vicinity of the dam are connected to stable datum points located in a place, farther downstream so as to be outside the area, influenced by the subsidence caused by dam and reservoir load.

From each of the paired theodolite piers near the dam, accurate angular measurements are made between each target and the opposite theodolite pier. These measurements are related by triangulations to measured stable base line. Difference between successively measured position of each target furnishes a path of target as it occurs due to the deflection of dam. Component of deflections in desired direction are then determined by resolution of these measurements. By judiciously choosing these points, accurately measuring base line length and verifying conservations of the scale and the orientations of the angle measuring device, by appropriate means precision ranging from 0.5 to 1.0 mm can generally be attained in determinations of the absolute positions of the absolute marks and from 0.2 to 0.4 mm in the measurement of their relative positions. To achieve such precisions many precautions must be taken and extremely skilled operators are required.

The geodetic method has the advantage of measuring absolute displacements with a high degree of accuracy but it involves complicated calculations which require considerable time.

On account of elaborate system of measurements, the great many precautions required to be observed to ensure the specified accuracy, precise measurement of base line, considerable time is taken to complete one set of observation during which period significant fluctuations in the reservoir level occur which inevitably affect the accuracy and impair the reliability of these observations. The above-mentioned reasons and the laborious calculations involved to process the observed data to obtain information on the dam displacements prevent the triangulation method being annually repeated a number of times that is sufficient and necessary to provide an accurate check on the behaviour of the dam. This is particularly true when the actions exerted on the structure change rapidly.

A variation of geodetic method combining the requirement of accuracy and speed, making subsequent measurements independent of and unaffected by changes in the base line lengths, to make it applicable for desired frequency of observations developed and applied by Federal Topographic Survey at Wabern Berne Switzerland, is described in the paper "*Investigation of Spatial Deformation in Power Dams by Geodetic Methods*".

B. Geodimeters: Electronic optical apparatus has recently been developed and used for measurement of small displacements (distance changes) and deformations as mentioned under para 2.1.1.1. Variety of such

equipment under different name is available in the market. However, operating principle of one of such apparatus named AGA Geodimeter, manufactured by Swen Ka AB Gasaccumulative is described as under:

The Geodimeter projects a pulsating light beam to reflector which returns the beam to the instrument. The projected beam sent out by the Geodimeter is modulated by means of high frequency signal applied to a Kerr Cell which serves as an extremely rapid electronic shutter.

The reflected light is received and converted to electrical signals by a phototube, the sensitivity of which is also controlled by high frequency signal and adjustable electric delay line inserted between the Kerr Cell and the phototube makes it possible to determine the phase relations between the transmitted and the received light pulse. When the correct adjustment has been made a null reading is obtained on a special instrument. The delay line readings are used for the computations of the distance. By using special measuring technique mean errors of the order—2 mm can be maintained over long intervals using the model-4 AGA-Geodimeter. For details on the apparatus specifications, operations and measurements range and accuracy and limitation of these apparatus, literature from the manufacturers may be referred to.

Targets: Selection of the type of target, its shape and size and face pattern suitable for observations is to be made after a number of trial designs and field observations by the surveying personnel from consideration of clear visibility, sharp intersection, distortion of its projection from distance, the exposure conditions and whether the survey work is to be carried out during day or night and accordingly necessity for lighting or painting with luminous phosphorescent paint. Target with conical top painted with contrasting paint and fitted with a white luminous screen have been in use on account of the fact that projection of cone is a triangle of identical dimensions whatever the direction of line of sight whereas plane targets have to be oriented properly to be normal to the expected line of sight for the expected programme of measurements but these still suffer distortion. Target with Bull's eye design *i.e.*, white on black background or black on white background, as also targets of other variation of face patterns have been in use (ref. page 48 of "European practice in Design and Construction of Concrete Dams Vol. III U.S.B.R.").

Location of Targets: Targets should preferably be located in grid patterns at locations which define the horizontal (arch or beam) and vertical (Cantilever) elements of structure as adopted for theoretical analysis as this arrangement would provide direct assessment of the actual deformations/displacements for comparison with those obtained from analytical method (Trial load and Twist Analysis or model testing) without the need for interpolation.

Another consideration in the location of target on the downstream face of the dam is that these are located in sections containing plumb line and embedded

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instruments and at elevations corresponding to the observations recesses of plumb lines. Targets on the foundation abutments should be located in continuation of the grid pattern on the downstream face of the dam so as to have a few targets on either abutment to cover an area expected to be potentially affected by dam and reservoir load.

To obtain well defined deflection curves a minimum of five targets should be fixed at each of the horizontal section suitably spaced, these targets being so placed as to define 5 vertical sections to provide well defined vertical deflection curves for such vertical sections.

This arrangement may be adopted in case of small concrete or stone masonry dams, or hollow gravity dams, buttress dams or multiple arch dams. Consideration of intervisibility from the concerned instrument station will influence location of targets near the toe rock-concrete contact and on the abutment.

Theodolite Piers : Theodolite piers may be one foot square, reinforced concrete prisms about 42 inch high, rigidly dowelled into the foundation rock or a supporting concrete structure.

Fixed to the top of each theodolite pier is a stainless steel plate which may be 9-inch in diameter and 3/4 inch thick which contain 'V' grooves spaced 120° apart for positioning the vertical axis of a theodolite so that it passes through the centre of the plate. The tripod is removed from the theodolite so that the instruments foot screws may be placed in this pier plate grooves. Instrument levelling is done directly on the pier plate. Each pier plate groove and each levelling screw positions on the theodolite is indexed so that the instrument may be placed on a pier plate in the same position for all measurement, thus eliminating the problems of centering the instrument on the pier plate each time it is used.

At the centre of each pier plate a tapped hole (1/2 inch diameter) is provided into which a pin will fit. The end of the pin protruding above the pier, is threaded to accept a sighting target. Each pier should be provided with its own sighting target as a further expedient of automatically eliminating error in the set up.

In the case of Wild T3 theodolite centering bolt is provided as part of the accessory for above expeditious set up.

Location of Theodolite Piers : Location of theodolite piers to be such as to provide at least three instrument stations on either side of the river downstream of the dam for sighting on each of the target providing such intervisibility to a maximum number of targets, as also intervisibility with the reference instrument stations (Base line Stations) and well conditioned triangles, formed by line joining the pair of instrument stations and respective target positions. Positions of the instrument stations are fixed by study of topography in the vicinity of the dam in the field and by adequate experimentation in respect of target shapes, sizes, lengths of sights, considerations of founding the piers on firm sound rock and other consider-

ation mentioned above, with a view to establishing telemetric measurement net capable of providing the needed accuracy and reliability of measurements combined with speed in execution.

Base Line : Base line is essentially a line joining two reference stations (datum stations) at each end of the line located downstream of the dam on firm and level stretch of ground assumed to be free from the deformations caused by dam and reservoir load with which all the triangulation measurements are connected for determining the absolute positions of the targets.

The accuracy and reliability of the entire series of triangulation work depends upon the stability of the base line length during the period of such series of measurements. As the distance between the piers (base line stations) changes as a result of ground subsidence and variations in rock temperature and moisture content, these distances are measured at least once during a complete series of triangulation measurements. As a normal procedure Invar wire or tapes are used for this purpose. To ensure stability of the base line throughout the durations of measurements during which time reservoir level fluctuations occur, it is imperative to locate it in area not subject to subsidence due to dam and reservoir load.

On Italian Dam, measurement of changes in distance between primary piers is made by means of basi meters (for details ref. pages 45-47, European Practice in Design and Construction of Concrete Dam Vol. II).

Bench Marks : For measurement of the vertical component of displacement by precise levelling, bench marks which may be stainless steel or bronze markers (points) are provided in the galleries of the dam, and in the tunnels in abutment as well as on the abutments on either side of the river. Precise levelling by using Wild N3 precise level and levelling rods is periodically carried out from stable reference points (bench mark) to these points for determining the variations in the level of these points as affected by the imposed dam and reservoir load. Lines of bench marks may be set farther away from the dam-concrete rock contact for measurement of the regional settlements.

Observations : Observations for measurement of horizontal displacement are made by mounting Wild T3 Theodolite on each of the paired theodolite piers and making accurate angular measurement between each target and the opposite theodolite piers in accordance with the first order triangulation requirements.

Frequency of Observations : Precise levelling operations on bench marks are carried out monthly unless some sudden significant changes in bench marks manifest when more frequent observations may be made.

Angular measurements for determination of horizontal displacement of targets by triangulations and of traverse marker by precise traversing should be carried out at times when the lowest and highest reservoir levels prevail and for one or two intermediate

levels during initial filling and in subsequent filling until the conditions stabilise when readings corresponding to the maximum and minimum reservoir level may continue. If apparatus of specified reliability and accuracy become available for precise measurement of the distance change *i.e.*, displacement of dam points as by Geodimeter, more frequent schedule of measurement may be adopted.

Forms of Records: Measurements over the system of theodolite piers downstream of the dam should be recorded on printed forms standardized for the respective project. Typical samples of such forms are shown as Fig. II-3.1-01 to II-3.1-03.

Sources of Error : There are three principal sources of error which influence the accuracy and hence the reliability of the triangulations measurements.

1. Errors due to instruments.
2. Errors resulting from apparatus.
3. Errors attributable to environmental influences.

The first group comprises the following:—

- (a) Error due to imperfect verticality of the principal axis of the theodolite.
- (b) Error due to graduations of circle.
- (c) Error due to optical micrometer.
- (d) Errors of the observing personnel.
- (e) Instrumental errors in measurement of the base line.

The following errors are covered under second group:—

- (a) Error due to eccentricity of the target on the pier.
- (b) The errors attributable to eccentricity of theodolite on pier.
- (c) Imperfect verticality of the target axis.

The third group comprises the following errors:—

- (a) Error due to dam movement during the period of measurement.
- (b) Errors due to diurnal periodic movement of piers.
- (c) Errors of phase.
- (d) Errors due to lateral refraction.

To eliminate errors caused by refraction, readings are taken in the morning and late afternoon or on cloudy days.

Readings may be taken at night using powerful searchlights. The theodolite should be protected from the sun and rain by means of a beach type umbrella or a permanent enclosure.

Method of Analysis: The measurements are adjusted by solving the normal equations by the method of least square and the final resulting values resolved

into the two required directions—radial and tangential—in the case of arch dams and normal and parallel to axis of dam, in case of dams having straight axis.

3.1.1.2 Measurement of Displacements by Plumb Lines

Plumb lines provide the most effective, simplest and quickest method of measurement of horizontal displacements of points inside the dam. Two types of plumb lines are in use namely (i) Regular plumb line, also designated as pendulum and (ii) Reverse (Inverted) Plumb line.

Regular plumb lines are used for measurement of the relative horizontal displacement of points inside the dam, relative to a reference point in the lowest gallery, available closest to the base of the dam. As it is not advisable to install the permanent deflection measuring equipment for plumb line observations, while the construction is in progress, on account of the great likelihood of damage to such costly equipment, it is usual practice to install temporary plumb lines for observations during constructions and until such time that the permanent deflections measuring equipment is installed.

(i) Principle and Construction

Plumb line essentially consists of a rust resistant wire suspended from the top of the plumb line well (shaft) provided for the purpose in the body of dam, by some suitable suspension arrangement and carrying a heavy plumb bob (weight) immersed in a damping medium contained in a can provided at the bottom of the plumb line well. Observation recesses are provided at the bottom and at suitable intermediate locations of the plumb line well (shaft) wherein are fixed deflection measuring equipment for observations of position of the plumb wire with respect to fixed reference mark in the observation recess. The observation may be made by directly reading the position of the wire against scales fixed to the observation recess walls, as in the case of temporary plumb lines or by using optical apparatus namely micro-meter-micro-scope, Huggenberger Coordi-scope, or Huggenberger Coordi-meter. Differences of the observed values of wire position from the reference readings (initial readings) when processed, provide values of the dam deflections at the levels of observation recesses relative to the lowest point of the plumb line which is considered to be fixed.

The various components that constitute the plumb line, temporary or permanent and equipment for observation of dam deflection, are as under:

A. Plumb Line Assembly

(i) *Shaft:* The shaft for housing the plumb line wire should be vertical formed hole of about 10" diameter. Where the shaft is located closer to the reservoir face of the dam, it is preferable to line the shaft. Observation recesses at gallery locations are provided and fitted with air-tight steel doors to prevent draft effect and unauthorised tampering of wire, when installed.

(ii) *Suspension Assembly*: It is usual to suspend the plumb line wire at the upper end of the shaft by means of a collet and nut in the centre of a heavy steel suspension spider (shown in Fig. II-3.1-04) placed over the shaft opening.

(iii) *Wire, Plumb Bob and Dashpot*: Stainless steel wire of about 0.03 inch dia may be used for the plumb line.

Solid brass cylinder weighing about 30 pounds or so may be used as plumb bob. The cylinder dimensions should be such as to keep it immersed in non-corrosive damping oil contained in galvanised steel container for damping possible vibration of the wire.

Suitable arrangement to prevent moisture condensation from falling into the oil, should be made, where it happens to be excessive.

B. Deflection Measuring Equipment

The following items comprise the above permanent equipment where observations are to be made by using micrometer-microscope. The arrangement has been designed by U.S.B.R. (USA). For temporary plumb line any arrangement providing for measurement of plumb line position against scales fixed in the observation recesses parallel and normal to dam axis is suitable. The scales should be rust-resistant and graduated in inches with 1:50 scale.

(i) *Reference and Microscope-Support Bars*: Within each observation recess two vertical 1/2 inch thick plates welded to form a right angle are placed in a position to contain the plumb line within the 90° quadrant formed by the plates. A stainless steel micrometer-microscope support bar and reference bar each extend at right angle from each of the above plates, as shown in Fig. II-3.1-04.

(ii) *Micrometer carriage and microscope*: As shown in Fig. II-3.1-04 carriage is constructed in such a manner that its base slides over carriage rod and anchored by cone-point screws seating into drilled recesses in the rod. Lead screw knob provides for reading to the nearest half thousandth of an inch. The carriage should have a total travel range of two inch.

Microscope of 20-22 Power and focussing through 1 1/2 inch having cross hairs in the eye piece has been found suitable for observations of plumb lines in observation recesses detailed in Fig. II-3.1-05.

Huggenberger Coordi-scope consists essentially of a micro-meter micro-scope complete with the micrometer carriage, and 90 degree circular quadrant track for positioning the micro-scope for observing plumb line movement normal and parallel to the dam axis. For using Coordi-scope, its base plate is fixed to the observation recess face by means of a bolt and so oriented as to enable measurement of plumb line movement in specified direction; Coordi-scope is mounted on to this fixed base plate by screwing on its base socket on to the threaded support point on the

base plate and seating it against the lower support points provided on the base plate.

The base plate is provided with a cover for its protection when not in use. The Coordi-scope has a range of 100 mm and accuracy of .02 mm.

Huggenberger Coordi-meter permits determination of the dam deflection at the location of observation recesses by processing of observed data consisting of angular changes from the original angular values. For details of the Huggenberger coordiscope and coordi-meter literature from the manufacturer should be referred.

(ii) Number, Location and Layout

In large concrete dams built, in V shape canyon, a plumb line should be provided in the block representative of deepest central portions and a plumb line each at quarter point of the length of the dam, for reading dam deflections of blocks built on steeply sloping abutments. In the case of dams built in wide canyon, such that the dam blocks are almost of the same height for most of the length of the dam, one plumb line each at one third the length of the dam, would be adequate for providing the necessary information on dam deflection behaviour.

Additional plumb lines may be provided in blocks founded on rock formations composed of strata susceptible to differential settlement when subjected to the operative loads.

In the case of buttress dams provisions of at least one plumb line should be made in the tallest buttress.

Location and Layout: Plumb line wells are usually located closer to the upstream face but the exact location will depend upon the locations of longitudinal galleries in the dam located at various elevations, for inspection and maintenance facilities.

The plumb line should be located so as to provide longest vertical continuous plumb line and the greatest possible number of reading stations with short access adits, if necessary, from that gallery to the reading station. In massive concrete gravity and arch dams, long plumb lines with adequate number of reading stations are easily available on account of the large number of longitudinal galleries provided in them. Typical example of such plumb line arrangement is shown in Fig. II-3.1-05.

However, in the case of large double curvature arch dams, having considerable vertical curvature, one continuous plumb line well extending from near the top of the dam down to the lower most gallery, may not be possible, if requirements of its proximity to the upstream face were to be adhered to. In such cases two plumb line wells are provided, one for the upper half of the dam and the second for lower half, such that the plumb bob elevation of the upper line is at the level of the suspension point of the lower line, located

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some distance upstream or downstream of the location of the lower half. The plumb bob and suspension point for the above two respective portions are housed in the same gallery or adit available in dam at that location. In the case of Valledi Lei Dam (Switzerland) (Ref. Vol. II "European Practice in Design and Construction of Concrete Dams U.S.B.R." Fig. III) three plumb lines are installed in the crown cantilever section in a stepped fashion.

In some cases where a single line from the top of the dam to the base is required, as in medium height doubly curved dam, the plumb line shaft extending from the crest intersects the d/s face. From this point down, the line is enclosed in concrete pipe.

For dome-shaped dams, the line is not enclosed. However, housing is provided at the toe of the dam for the bob or weight and the damping medium. In the case of buttress or hollow gravity dam, plumb line well is fixed on the side of the buttress.

Even in the case of the massive concrete gravity or arch dams, the location of longitudinal gallery near the top of the dam and closer to the upstream face than the locations of longitudinal galleries lower down, would necessitate adoption of two plumb lines with staggered locations so that deflections of the entire dam profile may be determined.

Layout of the observation recesses (reading stations) accessible from the longitudinal galleries through access adits should be such as to permit installation of deflection measuring equipment in such orientation as will enable direct measurement of horizontal dam deflections, parallel and normal to dam axis (tangential and radial directions respectively in case of arch dams) precluding any trigonometrical computations for the analysis of observed data. Typical layout of observation recesses at intermediate reading stations, at the suspension point and at the plumb bob locations is shown in Fig. II-3.1-05.

When formed and unlined shaft (plumb line well) has been provided, care should be taken to line all such portions of the shaft which are just within a few inches of the gallery face.

(iii) Method of Installation

A. Deflection measuring equipment for use with micro-meter microscope:

(1) *Recess fixtures installation*: The recess fixtures should be assembled as shown in Fig. II-3.1-04. Drill holes for expansion anchors and reinforcing dowels in the recess concrete. Drilling of these holes should be done carefully to ensure desired accuracy in alignment. Alignment may be maintained by using a small wood jig, set on the floor of the recess to hold the fixtures level and parallel to the faces of the recess. Exact alignment is to be obtained by small transit so as to set up and turn 90° in galleries.

Grout the anchors and steel dowels in place when the fixtures have been oriented to desired positions. Make final check for carriage rod alignments and then encase this frame in concrete. Install permanent plumb line as described below:—

(2) *Suspending the Plumb Line*: The permanent plumb line should be located at or near the centre of the net opening of the shaft, which should be determined suitably by suspending a transit plumb bob on a string or cord of a length sufficient to reach from the top of the shaft to the lowermost point.

Kinks and twists in the wire should be avoided when installing the same. The wire previously wound on a spool in a reeling rack is threaded through hole in the centre of the plumb bob plug, removed previously, the interior of which contains a recess. The wire is twisted around a short nail and hot solder poured in to cover the nail and fill the recess. After the solder has cooled, the plumb bob plug is lowered through the shaft to the lower most point. When the plumb bob plug is at the level of the damping pot, the plug is screwed into the bob, lowered into the damping pot and sufficient oil added to cover the plumb bob. The freely suspended bob is adjusted to an elevation just below the oil level in the dashpot, and the wire permanently fixed at the suspension point. The closing nut of spring collet is tightened to securely grip and hold the plumb line wire.

(3) *Marking reference points*: After installation of the plumb line, plumb bob, and placement of the plates is completed a permanent reference mark is scratched on the 3/16 inch dia stainless steel plug fixed in the flat face of each marker rod, care being taken to scratch the line vertically against the prevailing position of the wire.

(iv) Observations, Form of Record etc.

— (A) *Technique of Observations*: Set the micro-meter carriage and micro-scope assembly on a carriage rod and clamp the entire assembly in a position such that the reference mark and the plumb line fall within the range of the micrometer slide.

2. Focus eye piece sharply on the system of cross hairs.

3. Focus objective lens on the marker rod. Move the slide and microscope to the left of the reference mark and then by means of the microscope slide wheel, bring back the slide until reference mark centered at the cross hair intersection, and record micrometer slide position.

4. Repeat operation approaching reference mark from the right side, record micrometer slide position.

5. Focus objective lens on the plumb line and move slide and microscope to the left, bringing it slowly to coincide the cross hair intersection with left-edge of the wire. Continue movement of the slide till cross hair pattern is centered over the right-edge. Read and record micrometer slide position.

6. Repeat operation approaching plumb wire edges from right side and read and record micrometer slide position.

7. Difference of the average of step 3 and 4 readings and average of step 5 and 6 readings represent the position of the plumb line wire, with respect to the reference mark.

8. Repeat steps 3 through 7 twice more from the same support bar.

9. Repeat steps 1 through 8 for the second pair of carriage and marker rod.

(B) *Frequency of Observations*: Weekly observation should be made during the first filling of the reservoir after which fortnightly schedule of reading may be adopted. More frequent schedule of reading may be adopted when rate of reservoir rise is fast.

At least one reading must be obtained for the highest reservoir level attained every year.

(C) *Form of Record*: Data observed from the plumb line should be recorded in printed form shown at Fig. II-3.1-06 which should be got printed sufficiently in advance of the time of commencement of the observations.

Two copies should be prepared, the original be sent to the design office or officer responsible for processing observed data and the other copy retained at site for reference in case the original is lost in transit.

(v) *Analysis of Data*

Magnitude of dam deflection at any particular observation recess location relative to the lowest point of plumb line is obtained by subtracting the magnitude of movement of plumb line wire at that level from that obtaining at lowest level of the plumb line wire. Deflection values so computed should be plotted to show the deflected shape of the dam. Continuous plots of deflections at the various observation recesses, with corresponding reservoir level plot should be maintained for watching the deflection trends and behaviour of the dam.

Plotting of dam deflections against reservoir levels should be plotted for the suspension points.

3.1.1.3 *Displacement Measurements by Collimation (Alignment Survey)*

(i) *Principle and construction*

Collimation techniques provide one of the simplest and most effective surveying methods for rapid determinations of the position of a dam crest. Two basic steps are required to measure deflections of the crest of a dam by collimation. First a line of sight is established which essentially remains fixed in space during the life of the structure. Second the movements of points on the dam crest with relations to the line of sight are determined.

The line of sight passes through two reference points located on the right and left abutment rock. The elevation of the points is approximately equal to the elevations of the crest of the dam. So that the reference points will not be subjected to ground movements, resulting from the weight of the dam and reservoir, they are set back some distance (about 50-100 feet) from the dam rock contact.

A concrete instrument pier is constructed at the right and left abutment reference points. For protection against the weather and the direct rays of the sun, a permanent shelter is usually constructed over each pier.

To establish the line of the sight a collimator is set up on one pier and sighted in on a target, mounted on the top of the pier located on the opposite abutment. When this reference has been fixed, the telescope is rotated in a vertical plane until the horizontal cross-hair is on the centre of a movable target located on the crest of the dam. This target is moved by means of micro-meter screw so that its centre lines up on the vertical cross-hair. During this last step communication between the instrument man and the person adjusting the target is accomplished by hand signal or by telephone.

Collimations of all targets lying in a vertical plane through the line of sight between two companion piers is done from each pier. Usually more weight is given to the reading taken on the target nearest to the collimation.

To obtain the absolute movement of the crest the reference points are connected, by triangulation, to the stable base line, located at a place, assumed to be free from the deformation, resulting from dam and reservoir load.

For arch dams, where more than two target locations are required, additional companion piers are constructed.

(ii) *Number, location and layout*

In the case of straight dams having axis on the upstream sidewalk of the road way, targets or markers made of steel or bronze may be set along such axis line in every alternate block, if not in every block. In the case of dam with curved crest, targets may be set on the centre of the crest.

Notwithstanding the above general consideration for location of target, care should be taken to locate targets in sections containing plumb lines.

The above general spacing may be modified to place targets at locations defining the cantilever elements in straight gravity or arch dams.

For more detailed descriptions of the method and the apparatus necessary for making these measurements, refer to pages 55 to 58 of the Corps of Engineers Manual EM 1110-2-4300. "Instrumentation for

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measurement of structural behaviour of concrete gravity structures" 1958.

3.1.1.4 Measurement of joint movements at Surface by Detachable Gauges

(i) Principle and construction

Measurement of joint movements internally at points inaccessible are made by remote control joint meters described under para 3.1.1.7. Measurement of joint movements at surface or at locations accessible from galleries is made by detachable gauges of the Whittemore (Fig. II-3.1-07) and Marion type (Refer pp. 603, Fig. 1 & 2 "Symposium on the Observations of Structures," Transaction Vol. II 1955 Lisbon). This is essentially a mechanical method of joint movement measurement.

For making measurements with detachable gauges, inserts have to be fixed in the concrete. Inserts may be of common steel or of stainless steel but stainless steel inserts are recommended, though they are expensive. The common steel inserts, however, also last well without rusting, if due care is taken such as constant cleaning and greasing.

The inserts are placed in relation to the joint so as to indicate opening and closing movements of the joint and any sliding movement of the adjacent block. For taking readings of the joint movement the gauges are placed on the inserts and the dial readings observed. Difference between the reference and subsequent readings gives the value of the joint opening available at the time of observation. However, these observed readings require to be corrected for temperature changes.

Inserts

Inserts for detachable gauges for observation of joint movements at surface, should preferably be of stainless steel, though those of common steel can also be used.

The inserts should be fixed in place after the concrete has been set by drilling holes, and the inserts placed in proper position in the hole and fixed with mortar. The inserts need special protection when placed in a pavement as shown in Fig. 6 of the reference, wherein is also shown the typical details for such inserts.

(ii) Number, location and layout

The inserts should be installed on the surface at locations, corresponding to the locations of joint-meters inside the dam and in blocks where joint-meters have been installed. These should also be fixed inside galleries across joints where these are accessible and at points corresponding to the joint-meters locations.

Where surface cracks are present and considered potential in influencing the structural integrity of the

dam, inserts should also be placed across such cracks for observation of crack behaviour under imposed loadings.

The arrangement for fixing the inserts across the joint is shown in Fig. II-3.1-07 (b).

Inserts may also be fixed in arrangement shown in Fig. II-3.1-07(c).

(iii) Method of installation

The inserts should be installed at specified locations by drilling holes in the concrete carefully and cleaning the hole of all loose material. The insert should then be inserted and aligned so as to be normal to the surface of concrete in which this is being fixed, and the space around the insert backfilled with mortar.

Use of gauge should be made to aid in aligning the axes of the inserts, being normal to the surface in which they are embedded (Fig. 7 of the reference).

(iv) Observation, form of record and sources of error

(a) *Observation Technique*: Observations of the joint movement are made by placing the detachable gauge into the inserts and observing the dial gauge reading. Care should be taken to have the inserts well cleaned, before taking the readings so that the instrument pins are always placed in exactly the same position. At the same time the joint movement readings are taken, readings have to be taken on a standard base in order to correct the former, for the affect of temperature variations on the measuring instrument. The observations should be recorded in standard printed form (Fig. II-3.1-08).

(v) Analysis of data

Analysis of the observed data in the case of the inserts arranged as at Fig. II-3.1-07(b) is done as under :—

If readings at time 1 were l_1 and i_1 for the concrete and invar respectively, and for time 2 were l_2 and i_2 the variation in joint opening will be

$$x = (l_2 - l_1) - (i_2 - i_1)$$

Sliding of the adjacent blocks can be obtained by making calculations from the measurements, as these are not made in the direction of sliding. Monographs facilitate such computation.

Such monographs are also necessary for calculation of joint movements from observed data when the inserts are arranged as in Fig. 8(b) of the reference.

The horizontal movement is obtained from the equation

$$X = \sqrt{C^2 - Y^2}$$

$$\text{and sliding movement of } Y = \frac{A^2 - B^2 + C^2}{2A}$$

where A is the measured distance between inserts on line parallel to joint plane, B distance between upper two inserts across the joint and C distance between lower inserts across the joint.

3.1.1.5 *Foundation Displacement (Vertical) by Settlement Measurement Device*

(i) *Principle and construction*

The settlement measuring device essentially consists of a wire, the lower end of which is anchored at the bottom of a hole, drilled to such depth as to have its bottom in the foundation formation, as may be considered free from the deformation resulting from imposed dam and reservoir loading, while the other end passes over a frictionless pulley, fixed to gallery ceiling available close to the foundation, and supports a plumb bob of suitable weight, to keep the wire taut, a scale being installed in the gallery for observing the movement of the plumb bob resulting from the deformation experienced by the foundation. This device makes use of the principle that if the supporting pulley experiences a certain amount of displacement caused by foundation settlement, the plumb bob movement will be twice that magnitude. As the scale for observing the plumb bob movement is fixed in the gallery it undergoes vertical displacement equal to that experienced by the foundation. The net movement of the plumb bob as observed on the scale thus represents the magnitude of vertical displacement of the foundation. A simpler arrangement for measurement of the vertical deformation of the foundation would be to install a rod (or a pipe) in the hole drilled into the foundation, whose lower end is anchored to the bottom of the hole, while the top end protrudes above the top level of the hole in the gallery, the rod being maintained in vertical position and free from contacts with the sides of the hole, and observing the movement of a certain reference point fixed in the gallery, relative to the rod top, the difference between the successive reading indicating the magnitude of foundation settlement. This arrangement possessing the apparent appearance of simplicity, however suffers not only from formidable difficulties in installations on account of long length of rod (or pipe) required to be maintained in dead vertical position without any intermediate supports, but also from the fact that such long standing rods (pipes) are subjected to considerable vibrations and temperature change which factors vitiate the reliability of the observations. The settlement measuring device, however, is free from these disadvantages, even though some corrections for temperature change may be necessary and this may be insignificant, as temperature variations in the foundation and galleries closest to it, are not significant. The components of the settlement measuring device are described as under :—

1. *Hole or Shaft* : The shaft or hole, required to house the wire and the necessary anchorage at its bottom, should be drilled dead vertically and should be of sufficient size so that the wire when installed, remains free from side contact, even when the hole experiences horizontal deflection, on account of foundation horizontal displacement, resulting from operating loads. The hole should be drilled deep enough so as to have its bottom located in foundation forma-

tions, free from the deformations caused by the reservoir and dam loads.

As it is not possible to drill dead vertically such deep holes, the size of the hole drilled, should be such as would make available a net dead vertical opening sufficient to meet the above-mentioned requirements. Holes having diameters of 5-inch or over should be adequate for installation of the settlement measuring device.

The hole should be provided with casing, so as to eliminate all possibility of damage to the wire.

Wire has to be of stainless steel and may be of 1/32 to 1/16" diameter.

2. *Anchorage* : Anchorage may consist of 1" or 2" diameter reinforcing rod about 5 feet long provided with collars and a tapped hole in the top about 1-inch deep, into which can be fitted a plug with centre hole and a recess in the bottom as shown in Fig. II-3.1-09.

3. *Plumb Bob* : The plumb bob may be of a shape and size and material as described under para 3.1.1.2 (I) A(iii) but should be equipped with a sight vane (Fig. 11-3.1-09) for ensuring horizontal line of sight when making observations. The wire and the plumb bob should be enclosed in steel cabinet as per item (5) below.

4. *Supporting Pulley* : This has to be a frictionless pulley mounted in a support for fixation to the gallery ceiling to permit verticality of the wire even when the support point experiences lateral displacements.

5. *Scales* : When direct sighting method is to be used for measurement of the foundation vertical settlement, scales of rust resistant material, preferably stainless steel, as considerable condensation is prevalent in the galleries close to foundation, and machine graduated in inches with 1:50 graduations should be used. The scales have to be installed at appropriate place for convenience in observation and also to permit their being enclosed in an air-tight steel cabinet provided to eliminate possibility of tampering of the equipment by unauthorised persons.

6. *Micro-meter microscope support bars and reference rods* : Where the observations of the settlement have to be made by micro-meter and microscope, a vertical support bar as described under para 3.1.1.2 (i) B (i) should be suitably installed vertically for mounting the micrometer microscope for observation. Also a reference rod on which reference mark, indicative of the initial position of the plumb bob as seen through sight vane is to be etched, should also be installed. This whole assembly including the plumb bob and scales if used instead of reference rod, pulley and supports should be enclosed in steel cabinet.

The micrometer-microscope supporting bar and reference rod have to be of the same specification as detailed under para 3.1.1.2 (i) B(i).

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(ii) *Number, location and layout*

Considerations governing the number of the settlement measuring device to be installed in a given dam will be governed by the following considerations:—

- (i) The character of the foundation formation of the dam.
- (ii) The dimension of the dam and type of the dam.
- (iii) Presence of any faulty and incompetent seams in foundation warranting special attention.
- (iv) Settlement measuring devices should be installed in sufficient number as to provide representative assessment of foundation behaviour under imposed loading.

In the case of massive high gravity and arch dams founded on formation expected to exhibit deformability characteristics, significantly different from those assessed from field and laboratory investigations and adopted for theoretical and experimental stress investigations, devices may be installed preferably in 100' × 100' grid, this spacing to be suitably altered where dimensions of the structure do not permit installations of sufficient number of instruments for establishing foundation deformation pattern across the foundation transverse profile. Five lines of such instruments with at least three in each line, if not five, should be the minimum provided in such high gravity and arch dams. In case of gravity dams built in wide U-shaped canyon where the blocks have almost the same height for almost the entire length of the gorge, two lines of three devices each, should be the minimum provided.

Additional instruments may be provided where special foundations conditions warrant. Installation of such device in the representative buttress, of a buttress dam or hollow gravity dam, near its heel and toe would provide useful information on the foundation deformability and yielding, resulting from possible load concentration if occurring under operation load.

(iii) *Method of installation*

A concrete pedestal of adequate size and height, for installation of the scales, sighting equipment, and the steel cabinet for enclosing the settlement measuring device and for convenience in observations, should be provided on the gallery floor, so located as to permit extension of the vertical hole through it in continuation of the vertical hole in the foundation drilled for installation of settlement device. The height of pedestal should be such as to permit sufficient clearance between its top and the pulley supporting the wire and plumb bob, to accommodate the expected range of the foundation settlement. For installation of the settlement measuring device, it is necessary first to establish the centre of the net vertical opening, available in the hole drilled into the foundation and

to mark point on the gallery ceiling, in continuation of the vertical centre line of the hole. Position of the cinch anchor holes for fixing the pulley assembly to the ceiling, should then be marked with reference to the point so as to permit verticality of the wire when finally installed. The pulley assembly should then be fixed to the ceiling by means of cinch anchors driven into carefully drilled cinch anchor holes.

Installation of the wire in the hole and over the pulley should be done carefully to avoid kinks and twists in the wire. The wire in sufficient length previously wound on a spool in a reeling rack is threaded through the hole in the centre of the plug of anchorage, removed before hand, the interior of which contains a recess. The wire is twisted around a short nail and solder poured in to cover the nail and fill the recess. After the solder has cooled, the plug is screwed into the anchorage and the whole anchorage assembly lowered into the hole until it "bottoms" the hole.

Hold the wire in vertical position in the centre of the hole, while the anchorage is at the bottom of the hole and pour sufficient quantity of grout so as to provide at least 3 feet of grouted depth at the bottom. When the grout is finally set, wire is pulled taut and passed over the pulley and the other end threaded through the plug of the plumb bob, the interior of which contains a recess, the plug having been removed from the plumb bob earlier. Adjust the wire length such that the plumb bob will be at the desired position when finally the plumb bob plug is screwed into the plumb bob. When the required length of the wire has thus been determined the wire threaded through the plug is wound around a short nail and hot solder poured in to cover the nail and fill the recess. After the solder has cooled, the plumb bob plug is screwed into the bob. The wire is thus finally installed for use as settlement measuring device.

The scales or alternatively the micrometer microscope support bars and reference rods are installed at the specified position in exact vertical position.

The whole assembly is then finally enclosed in steel cabinet, fixed by means of cinch anchors to the gallery walls and provided with internally locking arrangement to prevent tampering of the equipment by unauthorised persons.

(iv) *Observations, forms of record*

(a) *Method of observations*: If the scales are used, then the initial reading on scale corresponding to the level of the cross hair in the sight vane as observed by direct sighting should be recorded in the printed forms. Subsequent readings corresponding to the sight vane cross hair, observed at specified intervals should be noted and recorded.

When readings are taken by using micrometer microscope a fine line should be etched on the reference rod to indicate the initial position of the plumb bob

sight vane cross hair and initial reading of the micrometer in the microscope is recorded in the standard printed form. Subsequent reading obtained corresponding to the level of the plumb bob sight vane cross hairs by adjusting the micrometer when micrometer microscope is mounted on the support bar should be recorded in the standard printed forms. Sample of standard data form is at Fig. II-3.1-10.

(b) *Frequency of Observations*: Frequency of observations should be same as under para 3.1.1.2(iv)B.

(c) *Form of record*: Typical printed standard form for record of field data is shown in Fig. II-3.1-10.

v. Analysis of data

Computations to obtain the values of foundation deformations from observed data are simple. Difference of initial readings from successive observed values provides the values of foundation settlement obtaining at the time of a particular observation.

3.1.1.6 Measurement of Dam and Foundation deflection by reverse (inverted) plumb line

(i) Principle and construction

Use of regular plumb line for measurement of relative horizontal displacements of the points inside the dam, with respect to the lowest point of the plumb line, has been described under para 3.1.1.2.

The relative displacements are determined by considering that the reference point, namely the point corresponding to the lowest reading station, remains fixed and does not experience any shift in its position. In actual practice, however, the reference point itself undergoes horizontal displacement on account of the foundation horizontal displacement under operating loads. So in order to obtain correct evaluation of the displacements of dam points it is essential to ascertain the magnitude of foundation displacement and hence the shift of point corresponding to the lowest reading station resulting from varying reservoir level.

This measurement is made possible by the use of reverse plumb line, which essentially consists of a rust-resistant wire the lower end of which is attached to an anchor grouted to the bottom of drilled hole in the foundation while the suspension point (upper end) is attached to a float which is buoyed up by water in a covered tank. To reduce evaporation, the water is covered by a film of oil.

For measurement of the foundation displacement (horizontal) for applying correction to displacement measured by the regular plumb line to obtain correct values of horizontal displacement of dam the suspension point and float arrangement may be installed in a gallery in which the plumb bob and damping medium for the regular plumb line is housed.

The arrangement should be so positioned as to permit observation of the reverse plumb line at an elevation corresponding to the lowest reading station elevations of the regular plumb line.

However, if the reverse plumb line is extended all the way upto the topmost available observation station, values of the absolute horizontal deflection of dam points become directly available by simple difference of the initial and successive micrometer microscope readings if the microscope or co-ordiscope is used or difference of initial and successive reading of scales against wire position if direct sighting method is used, requiring use of scales.

The items of equipment and other features required for installation and use of reverse plumb line. Typical installation of which is shown in Fig. II-3.1-11 as under:—

1. *Hole*: Hole has to be of sufficient size as to permit free positioning of wire in it, even when the hole experiences horizontal displacement on account of foundation horizontal displacement and of such depth as to anchor the lower end of the wire at points in the foundation considered free from deformations due to dam and reservoir loads. The hole should be sufficiently vertical as to provide a net vertical opening of about 8 inches in diameter.

The hole has to be cased but for the bottom 3-5 feet which portion is filled with grout for anchoring the lower end of wire attached to an anchorage unit.

2. For description of wire, micrometer microscope, reference and support bars, suspension arrangement refer to para 3.1.1.2.

3. *Float*: Float is essentially a double walled hollow cylinder of steel covered at top and bottom but having a central hole through the top and the bottom cover for threading the plumb wire from below and connecting it to the suspension collet fitted in a suspension spider centrally supported on top of the float. The central opening in the float should be of sufficient size to accommodate horizontal displacement of the dam at float level without contact with the plumb wire.

The size and height of the cylindrical float has to be designed in accordance with the buoyancy principle to provide the specified level below the top of the float as also below the top level of the cylindrical pipe welded in the centre of the tank bottom to provide continuation of the plumb line opening for extension of the plumb wire through it.

4. *Tank*: The tank is cylindrical in shape and may be fabricated from steel. An upright cylindrical pipe of the same size as that of the plumb well should be welded centrally over the hole in the centre of the tank bottom, to provide continuity of the plumb well for extending the wire to the suspension point on the float. The height of the upright pipe should be such as to preclude over-topping when tank is filled with

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water to such specified level that the resulting position of the float in the water would cause the specified amount of tension in the plumb wire.

The tank should be provided with air-tight cover to prevent evaporation of water and tampering of float and suspension assembly. Suitable arrangement should be provided for lifting the tank for repositioning of the tank in case the wire gets into contact with the above-mentioned upright cylindrical pipe.

5. *Concrete Pedestal*: Concrete pedestal of adequate size and height should be provided in the gallery for supporting the tank and float assembly ensuring vertical continuity of the plumb wire. The height of the pedestal should be sufficient to accommodate observations recess for installation of deflection measuring equipment for observation of the plumb wire.

6. *Anchorage Unit*: This may consist of $1\frac{1}{2}$ " - 2" diameter reinforcing steel bar, about five to six feet in length with collars welded to it at suitable spacing to provide the desired anchorage action when grouted in position. Hole of suitable size and depth should be tapped centrally in the top portion of the rod for attachment of plumb wire.

(ii) *Number, location and layout*

For this refer to para 3.1.1.2(ii).

(iii) *Method of Installation*

For installation of deflection measuring equipment, micrometer microscope support bar, reference rods etc. and suspension assembly refer para 3.1.1.2(iii). For installation of the reverse plumb line, stainless steel wire, sufficiently in excess of the length, required for any specified location is wound on a reeling rack and the free end of the wire is passed through the hole in steel plug as mentioned under para 3.1.1.2(iii) and tied to a small nail, and pulled tight. Molten lead or zinc is poured into the recess of the plug. After the zinc has set, the plug is screwed into the tapped hole in the top of anchorage. The whole anchorage unit assembly is then lowered into the hole and positioned so that wire occupies central position of the net vertical opening of the hole. Required quantity of grout is then poured into the hole, while keeping the anchor and wire in central position so that about 5' depth from bottom of the hole is filled with grout.

After the grout has set, grouting of the annular space between the casing pipe and the inside of the hole should be done.

The rest of the procedure of threading the plumb wire through collet and supporting the suspension spider is the same as described under para 3.1.1.2(iii) with the only difference that in this case spider rests in the specified position on the float top instead of the concrete pedestal.

(iv) *Observations, Forms of Records etc.*

Procedure for observation is similar to that described under para 3.1.1.2(iv).

Forms of record same as specified under para 3.1.1.2 (iv).

(v) *Analysis of data*

Differences of the readings at respective reading stations from the respective initial reading at corresponding reading stations will give the magnitude of the deflection at the level of the respective reading stations.

3.1.1.7 *Measurement of internal joint movement by joint-meters*

Depending upon the principle of working two kinds of joint-meters for internal joint movement measurements are in use:

1. Carlson elastic wire resistance type.
2. Vibrating wire type (manufactured by Telemac and Maihak).

(i) *Principle and construction*

1. *Carlson elastic wire resistance type joint-meter*: The meter is in the general form of a long cylinder with anchor on one end and a threaded hub at the end which fits into the tapped end of a socket intended to be cast in advance in an adjacent block, to engage the surrounding concrete. (Fig. II-3.1-12).

In order to provide for an elongation of as much as 0.20 inch long single loops of steel wire are held in tension by small coil springs and pulleys arranged within the flexible brass cover tube. Since tension in the springs is directly proportional to displacement this arrangement makes it possible to keep the proportionality of stress and consequently resistance in steel wires to the overall displacement. The greater part of the displacement is taken up in the coil springs.

The instrument is designed to take advantage of two electrical properties of steel wire, resistance varies directly with temperature and resistance varies directly with tension. When the ends of the joint-meter are pulled apart by opening of a joint, the outer or expansion loop elongates and increases in tension and consequently in resistance as well. At the same time the inner loop or contraction loop decreases in resistance as it shortens. The ratio of the resistance of the expansion loop divided by the resistance of the contraction loop is used as a measure of joint opening. Temperature is measured by taking the sum of the resistances of the expansion and contractions coils.

Since there may be considerable shearing movement in contraction joints as adjacent blocks cool at different rates the joint-meter is designed to withstand a

shearing movement of 0.10 inch by reducing the frame section at the ends to serve as elastic hinges. A flexible bellows forms the centre section of the joint-meter to take the longitudinal and shearing motions of the case without undue straining.

The meter is usually furnished with 30 inches of rubber covered, three conductor cable attached, the conductor having coloured insulation of green, white and black respectively.

The green coil is made common to both the loops. The white one goes to the end of the expansion loop and the black one goes to the end of contraction loop.

The loops are encased in brass tubing which is completely filled with corrosion resistant oil. A separate sealing chamber is provided for terminating the rubber covered cables that connect the joint-meter to the terminal boxes in the galleries.

The calibration of the joint-meters are about .0005 inch joint opening for every 0.01 per cent increase in resistance ratio. The resistance of coils varies about 1 ohm for each 11°F rise in temperature. Measurements of the resistance ratio and resistances are made by connecting the three conductor cable to the binding posts of a standard wheatstone bridge circuit test set in specified order in accordance with the manufacturer's instructions.

2. Vibrating wire type joint-meters (Telemac or Mahak make): These joint-meters function on the principle that change in length of a pretensioned steel wire vibrating with damped natural vibrations results in a proportional change in the natural frequency of the vibrating wire. The measuring system consists of a transmitter (transducer) containing a pretensioned steel wire (the measuring wire) stretched so as to vibrate in damped natural vibrations after excitation from a receiver and an electromagnet. The vibration can be started automatically or by hand by pressing a button arranged on the receiver.

When the wire is plucked, an alternating current signal is indicated in the magnet circuit. This signal has the same frequency as that of the vibrating wire. An increase in the tension of the wire, produced by the lengthening of the transducer element causes a corresponding increase in the frequency of the signal.

The receiver or a comparator contains a steel wire (standard wire) whose frequency is tuned to the frequency of the measuring wire by means of a micrometer. The adjustment of the standard wire is read on a dial, the difference in scale units multiplied by the calibration constant represents the direct result of the measurement. The mechanical natural vibration of both the measuring and standard wires are passed on to the Cathode Ray Tube of the receiver after having been transformed into electrical vibrations. The required coincidence of frequencies exist when the vibrations to be seen on the screen of the Cathode Ray Tube are united to form a circular or elliptical figure.

The transmitters are connected to the receivers with special electrical cables. Since the measurement being made depend upon the mechanical vibration of the steel wire, the fluctuations of electrical values (such as cable and contact resistances) in the transmission line do not affect the accuracy of the measurements.

(ii) *Number, location and layout*

Number of joint-meters required to be provided in any given dam will depend upon its dimensions, block layout, provision of transverse and longitudinal joints or transverse joints only, length of the dam, configuration of the foundation profile, presence of specially treated foundation features under the dam, and the extent to which measurements of joint behaviour would prove adequately representative of the joint movements for the entire structure. In the case of large massive concrete straight or arch gravity dams, transverse joints extending from upstream to the down-stream face and continuous from foundation for the entire height of the dams, divide the structure into blocks having transverse parallel sides (radial in the case of arch dams). Each such block is then sub-divided by longitudinal joints (circumferential joints in case of arch dam) into smaller monoliths the longitudinal joints being vertical and parallel to the axis (circumferential for arch dam) of the dam for the monolith entire height. Grouting of the longitudinal contraction joints in large gravity dams, subsequent to the cooling of mass concrete is an imperative provision to ensure structural integrity of each block and enable it to act monolithically to conform to the design requirements.

Grouting of transverse contraction joints in large massive straight gravity concrete dam is done to restore the entire structure to a monolithic condition to behave as a three dimensional structure and thereby introducing more favourable stress distribution pattern, specially beam stresses in straight gravity.

In the case of arch dams grouting of longitudinal joints (if provided) and of transverse joints is an essential requirement to weld the blocks into one monolith integrated structure conforming to requirements of design. In the case of stone masonry dams, joint movement measurement of transverse joints is required. To meet the above requirements joint-meters need to be installed in sufficient number and at locations which will provide information on the extent of joint openings and their groutability as representative of the joint system for the entire structure.

In the case of large dams built in V-shaped canyon, joint-meters should be installed in at least three blocks, namely one central block representing deepest and maximum section of the dam, and a block each in the abutment portions representing blocks built on steeply sloping abutments. In plan at a given elevation the joint-meters in each of these blocks should be installed at the centre of the longitudinal joints and at the centre of the transverse dimensions of the monoliths in the block, while these should be spaced about 50' vertically for the entire height of the longi-

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tudinal and transverse contraction joints as the grouting lifts (zoning of joints by provision of metal seal) are limited to about 50' in height. This spacing may be modified in the top portion of the joint if joint height does not permit of 50 ft. spacing for the entire height.

(iii) Method of installation

1. *Carlson elastic wire resistance type joint-meter*: Joint-meters are supplied equipped with a socket which must first be embedded in the high block. The socket is further provided with a slotted plug in which two holes have been drilled. Procedure for the installation of joint-meter at a contraction joint consists of the following steps:

(a) Nail the plug, slotted side out to the form at the joint-meter location. This should be not more than 6" below the top of the lift for easiest installation.

(b) Screw the socket on the plug.

(c) When socket installation is complete care should be taken by the workmen not to hang things on the socket or use it as a step in getting out of the form or mistreating it otherwise thus causing misalignment. To make the above encroachments it is a good precaution to run a few ties to the form to keep the socket in position Fig. II-3.1-13.

(d) If it is considered that for the particular concrete being used, the force exerted by the nails in the plug and ties is greater than the bond resistance on the socket, an additional anchor might be welded on to the socket, to prevent pulling it out during form removal.

(e) If the cable leads are to run in the block in which the socket is embedded, recess should be provided adjacent to the socket into which not less than 3 feet of the cable have been coiled.

After the low block reaches the elevation in which the joint-meter itself is to be embedded, the following procedure should be followed:

- (i) Let the placing foremen, complete the lift at the joint-meter location.
- (ii) Dig back at the location until the plug is uncovered, leaving a small trench about one foot square.
- (iii) Back out the plug and insert the joint-meter, screwing it upright.
- (iv) Tie the joint-meter cable lead out of the pour (if cable is embedded in the high block) and backfill with 3" maximum concrete, hand puddling the concrete around the meter.
- (v) Next day, splice the meter cable and the cable extension may preferably be done at the time of embedment to allow observation to be made.

2. *Vibrating wire type joint-meter*: Refer details in catalogue of the relevant type (Telemac or Maihak).

(iv) Observations, forms of record, sources of error

1. Carlson Joint-meter

(a) *Observations*: Observations of the resistance ratio and resistance of the joint-meters are made by connecting the meter conductors, until these are soldered to the terminal contacts on terminal boards in galleries, to the binding posts of the standard Wheatstone bridge test sets (Carlson) in the order specified by the manufacturer. Subsequent to their terminations on the terminal boards in galleries, observations of joint-meters are made by connecting the joint-meter terminations to the Carlson test set binding posts through standard test lead supplied by the manufacturer. The standard test lead is equipped with a female plug in socket at one end while at the other the individual conductors are equipped with metallic like terminations for connection to the test set binding post. Care should be taken to connect the cable leads in the order specified by the manufacturer to ensure correct and reliable measurements.

(b) *Frequency of observations*: The following reading schedule may be adopted:

1. Immediately after embedment.
2. Every 3 hours for the next 30 hours.
3. Every 12 hours for the next four days.
4. Once a day till the concrete temperature rises to a maximum (usually about two weeks).
5. Twice weekly for next one month.
6. Weekly afterwards until the completion of contraction joint grouting.
7. Fortnightly afterwards.

(c) *Forms of record*: Observations should be recorded directly on the printed field reading form (Fig. II-3.1-14) which should be got printed sufficient in advance of the commencement of observations.

Duplicate copy of observations should be prepared at the same time the observations are being recorded on the original sheet. This can be done by placing a carbon below the original one before commencing reading of the observations. The original should be sent to Design Office or to the office responsible for analysis of the data and the duplicate retained in the observer's office for future reference.

(d) Sources of error:

1. Presence of moisture on the terminal panels.
2. Grounding of the meters.
3. Loose circuit connections of the test set.
4. Faulty cable leads.
5. Presence of deposit on terminal contacts.
6. Low voltage of test set batteries as evidenced by galvanometer deflection.

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2. *Vibrating wire type joint-meter*: Observations of the joint opening are made by connecting the meters (transducers Telemac) or transmitters (Maihak) to the test sets or receivers (Maihak). As the connecting leads are only two conductor copper shielded cable no special order of connection is specified as in the case of Carlson type joint-meters equipped with 3 conductor cable leads. Readings are taken by plucking the transducer element (measuring wire) and adjusting the standard wire length by micro-meter movement until the two frequencies match as indicated by the appearance of figure of ellipse on cathode ray tube screen. The new reading of the micrometer is recorded in the standard printed form for further analysis of the observed data.

Frequency of the observation has to be same as in 3.1.1.7(iv) above.

Sources of error: Accuracy of the reading does not depend on maintaining high resistance to ground. The test set has electronic circuits and the components are affected by temperature and shock.

(v) *Analysis of data*

1. *Carlson joint-meter*: Analysis of data is done by transcribing the observed data to the "Standard Joint-meter Data Sheet" (Fig. II-3.1-15) and following the procedure detailed here under:

Fill in the calibration data of the meter as supplied by the manufacturer, in the space provided in the top portion of the form, against particular items. Compute the value of the corrected calibration constant by using the following equation:

$$C^1 = C \cdot \frac{YC}{R} (0.89)$$

Where C^1 is the calibration constant.
 C is the original calibration constant.
 Y is the resistance of a pair of conductor cables.
 R is the meter resistance at 0°F.

In column 3 is the observed resistance in ohms. In column 4 is change in resistance being difference from the reference value (zero value). In column 5 is indicated temperature computed as a product of (difference of observed resistance and meter resistance at zero degree) and change in temperature per ohm change is resistance.

In column 6 is the observed resistance ratio per cent. In col. 7 is the change in resistance ratio i.e., difference between the observed and the zero reading (when concrete sets). In column 8 is the indicated joint movement obtained by multiplying the change in resistance ratio by the corrected calibration constant.

2. *Vibrating wire type joint-meter*: The observed data should be transcribed in the standard printed joint-meter data sheets when received and analysis of

data to obtain values of joint opening done by using the calibration curve or the calibration constants supplied by the manufacturers.

3.1.1.8 *Foundation deformation measurement by deformation meters (joint-meters)*

(i) *Principle*

The principle underlying this arrangement is essentially to measure the deformation of the foundation surface with reference to point located deep into the foundation at depth considered free from deformation due to dam and reservoir loads.

The arrangement comprises in installing a joint-meter across the surface of contact of dam concrete and foundations rocks in a manner that its lower end is mounted on a pipe having its lower end anchored at the bottom of a drilled hole, the portion of the joint-meter in the hole being free from contact with its periphery while the upper half is embedded in the dam concrete.

Deformation of the foundations surface will manifest by changes in the resistance ratio of the joint-meter caused by movement of the upper half while the lower half is fixed. The change in resistance ratio from the initial reference ratio are indicative of the magnitude of foundation settlement obtaining at the time of observations due to imposed dam and reservoir loading. For the operating principle of the joint-meter, refer para 3.1.1.7(i).

The component of this arrangement are as under:—

1. *Hole or shaft*: 3" diameter vertical holes are required for this device. Depth of the hole has to be such that its bottom is located in foundation formations considered free from the influence of dam and reservoir load. The hole has to be cased for its entire length except for the bottom portion for about 2'-3'.

2. *Anchor and pipe*: 1½" × ¼" steel reducer for attachment to the bottom of the joint-meter to its top as per details to be described under para (iii).

3. *Joint-meter and socket*: For description refer to para 3.1.1.7(i).

(ii) *Number, locations and layout*

Refer para 3.1.1.5(ii).

(iii) *Method of installation*

After the hole of specified diameter and depth has been drilled and cased, as under (i) (1) above ¾" diameter pipe of required length with 1½" × ¾" reducer attached to its lower end is lowered into the hole until it bottoms the hole.

Joint-meter socket is screwed on to the pipe top while the pipe is maintained centrally in a vertical position as shown in Fig. II-3.1-16, the length of the

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pipe being such that the joint-meter when finally installed has its mid-section at the concrete rock contact. To break the bond of the socket from the periphery of the hole backfill the annular space between the hole and the socket with paper keeping the socket and pipe positioned centrally and vertically in the hole.

Keep the plug in the socket in position until commencement of concreting in the area of deformation meter location.

Remove the plug in the socket and screw in the joint-meter equipped with the required length of cable for connection to terminal boards in the galleries before concreting around deformation meter location begins.

Hand place concrete around the meter by removing all aggregates larger than 3" size and vibrate carefully with wiggletail vibrator so as not to damage the meter. Care should be taken not to dump concrete directly over the meter location. All other cares and precautions for installation are similar to those described under para 3.1.1.7(iii).

(iv) Observation

Refer to para 3.1.1.7 (iv) for procedure of observations, frequency of observation, forms of record and sources of error.

(v) Analysis of data

Analysis of data is done similarly as in para 3.1.1.7(v) with the only difference that here the results provide values of foundation deformations instead of joint movements (openings) as in para 3.1.1.7(v).

3.1.1.9 Measurement of tilt by clinometer

Measurement of tilt in dams essentially consists in measuring the change in angle of reference axes with reference to horizontal or vertical planes. These measurements are made in transverse or longitudinal galleries located at the base or directly below the crest of the structure. Tilt measurements at intermediate locations may also be made to provide as much checking data as possible not only to provide check on this deflections and settlements recorded by plumb lines and obtained by bench mark levelling respectively but also to supplement it where necessary. Instruments used for measuring tilt are called clinometers and inclinometers. Clinometers which measure changes in angles relative to horizontal planes, are more widely used than inclinometers. Two types of tilt measuring apparatus are in use:

- (a) Optical air-bubble type.
- (b) Electrical vibrating wire type.

(A) OPTICAL AIR-BUBBLE TYPE

(i) Principle and construction

The air-bubble type clinometer essentially consists of a precision level and a rigid frame. The level which is mounted at mid span is pivoted at one end and fitted with a micrometer screw at the other. Measurements of changes in tilt are made by supporting each end of the frame on reference points attached to the supports or bases which are embedded in the dam, and then centering the bubble by means of the micrometer screw.

The main differences between the various types of clinometers concern the methods used for supporting the instrument and for centering the bubble. One type of clinometer is mounted on two stainless steel balls, each of which is attached to a support bracket embedded in a gallery wall. The distance between the brackets is about 1 m (3.28 feet). To position the instrument when making rotation measurements each ball contacts a hardened steel button attached at each end of the frame. One of these has a cupped point and the other a "V" groove, with the axis of the groove parallel to the axis of the instrument.

The level is a large-radius vial containing sulphuric ether. When adjusting it, the customary method of positioning the bubble between two lines on the vial is used. A second level, having an axis set normal to the frame, is attached to the instrument. This is used to correct tipping of the clinometer about its longitudinal axis.

The second type of clinometer differs from the one described above in two respects. The first is that the instrument need not be held when read, since it has three points of support, one on one side, and two on the other side of the instrument. Each bracket also has three points of support. The outside point is cup-shaped, the middle point has a "V" groove, and the inside point is flat. By this arrangement of points, a clinometer can be oriented first in one direction and then in the other when taking readings on adjacent supports. All of the reference points on the clinometer are steel balls. The clinometer level is equipped with an optical positioning device which provides a double image for the bubble. The correct position of the bubble is attained when the boundaries of both images are made coincident. By means of this accessory, the position of the bubble can be set with greater precision. As a result, a level having a vial of less radius than the vial of the first instrument described can be used. This has the advantage of increasing the stability of the bubble.

Disadvantages are claimed for the two instruments described above. The first one is difficult to use, since it has to be held by hand when making readings. Readings made by the second one are subject to errors, since the support reference points used during normal and reverse positions when making clinometer measurements are not the same.

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In practice, clinometer measurements are made between several supports set in the concrete walls of longitudinal and transverse galleries. Normal and reverse readings are taken between one support and an adjacent support for each pair of points at a clinometer station. Measurements of rotation are subject to errors caused by movements produced by thermal effects and by volume changes in the concrete. The Huggenberger air-bubble type clinometer is supported on circular bases which is made up of three stainless elements, one with a hole, another with a slit, and the third with a plane on which the clinometer supports rest. The bases have the number of elements required to measure rotations in two orthogonal planes. Yet given the extreme sensitiveness of the instrument the supports must be very thoroughly cleaned as any dirt, even undetectable, can give rise to considerable error.

(ii) Number, location and layout

For providing information on settlement and checking of the values obtained by bench mark levelling observation, clinometer bases may be provided at locations corresponding to those of benchmark.

For measurement of change in angle of vertical plane, these may be provided in proper orientation at elevation where plumb line observations recesses have been provided. Refer para 3.1.1.2(ii).

(iii) Method of installation

The only part of this apparatus which needs installations is the base on which clinometer is supported for observations. The bases may be embedded in the gallery floors at specified locations by drilling holes of suitable depth and dimension depending upon the type and dimensions of the bases, installing the base and filling the space around with the grout. Care has to be exercised to ensure that the base is properly aligned and oriented and levelled.

(iv) Observations, sources of errors etc.

Observations of tilt are made by placing the clinometers on the bases in proper orientation and centering the air-bubble by means of the micrometer screw. The readings of the micrometer screw obtaining at any particular line of observations when referred to the initial reading gives the value of tilt.

Frequency of the observation is the same as for plumb line observations [ref. Para 3.1.1.2(iv)] for clinometers meant for measuring tilt from the vertical plane. For clinometers meant to measure tilt from horizontal plane, frequency of observations may be same as that of the benchmark levelling.

Sources of error.—As these instruments are very sensitive, presence of dirt, even when undetectable on supports affects the accuracy of readings. Temperature variations cause significant effect on the accuracy of observations.

(v) Analysis of Data

The observed values of tilt are processed to obtain the vertical and horizontal displacements of the points of clinometer locations by integrating the displacements beginning from the reference point.

(B) VIBRATING WIRE TYPE CLINOMETER

(i) Principle and construction

The measuring system comprises one receiver and any desired number of transmitters (transducers) which are connected by cables to the receiver. The transmitters are permanently fitted to the object whose change of inclination (tilt) is to be determined. The cylindrical case of the transmitter houses a special pendulum surrounded by damping oil with an extremely short pendulum length. A vibrating wire made of steel (the measuring wire) is stretched between the pendulum and the case. If the pendulum changes its position relative to the case, the mechanical tension and hence the natural frequency of the measuring wire is altered. An electromagnetic system picks up the mechanical vibrations of the wire, converting these into electrical oscillations which are transmitted to the receiver. The receiver also contains a steel wire—the standard wire—the mechanical stress and hence its natural frequency is tuned to the frequency of the measuring wire of the respective transmitter by means of a knob. The adjustment of this standard wire is read on a dial. The difference in scale units multiplied by calibration constant represents the desired result of measurement.

The mechanical natural vibrations of both the measuring and standard wires are passed on to the cathode ray tube of the receiver after having been transformed into electrical vibrations. The required coincidence of frequencies exists when the vibrations seen on the screen of the cathode ray tube are united to form a circular or elliptical figure.

Two types of electrical clinometers are in use.

(a) *Surface clinometers.*—These are to be fitted on the outside of the test object, for example to the vertical or horizontal surface of the structures. The rigid mounting is ensured by flanges or spreading anchors. In order to align the transmitters to zero position, during installation, transmitters are provided with a ballpin. The mounting flanges or anchors have matching ball sockets with locknuts.

(b) *Embedded type clinometers.*—These are meant to be installed in concrete or fitted to building or a construction part exposed to hydraulic pressure. The measuring element is arranged in a special protection bearing and may be aligned in the described zero position after concreting or locking the lower part of the housing. Both of these types are available for tilt ranges varying from 10' to 40' and having sensitivities varying from 0.3" to 2" and 1.4" to 10".

(ii) Number, location and layout

Refer. para A (ii) above.

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(iii) Method of installation

Details of mountings are available in the respective catalogues of the manufacturers of vibrating wire type clinometers.

The most well known makes are Maihak, Telemac and Galileo.

(iv) Observations etc.

Details for connections of the meter cable to the receivers (central recording board) are available in respective catalogues which should be referred to.

(v) Method of analysis

Calibration charts showing relationship between the change in frequency and the quantity being measured are supplied by the manufacturer. The observed data is therefore analysed with the help of these calibration charts.

3.1.2 Instruments for Measurement of Displacement in Earth Dams

Measurements of displacement in earth dams are made by optical, mechanical and electrical methods. Under the optical methods are grouped instruments for Geodetic survey for measurement of surface displacements, namely theodolite and levels for horizontal and vertical surface displacement measurement of surface points respectively. Ordinary methods of surveying are considered sufficiently accurate on account of large displacements experienced in earth dams in comparison with concrete and masonry dams, where precision instruments are used on accounts of comparatively smaller magnitudes of dam displacements and foundation deformations.

Mechanical methods of displacement measurement in earth dams are most extensively used for internal measurements. These include the use of internal vertical movements device cross-arm, and internal horizontal movement device and internal foundation settlement measuring device.

Electrical methods, not as extensively employed as the above-mentioned mechanical ones, involve use of electrical instruments.

The instrument fairly adopted for use for such purposes is the Wilson slope indicator.

Use of electromagnetic probe for measurement of vertical and horizontal displacements in earth dams is described in an article "Gepatsch Rockfill Dam in the Kanner Valley" by Lauffer H. and T. Schober, of Austria. Use of the Eastman gradient indicator for internal measurement of horizontal and vertical displacements of earth dam is also mentioned in the above article.

Swedish wells (floating concrete rings) have been used in Europe for measurement of internal movements of earth dams and embankments.

3.1.2.1 Instruments for measurement of surface displacement of embankments

Geodetic methods of embankment surface displacement measurement involve use of theodolite and level. The method consists in establishing surface point (markers) horizontal and vertical displacements of which are measured by standard surveying methods involving triangulation, traversing and levelling operations. Stable benchmarks and reference marks are established for reducing the observations to a reference datum. Detailed description regarding the considerations for layout and location of surface marks etc., their construction details, procedure for observations, accuracy of measurement and method of analysis of data are described in detail in designation E-32 pp. 707—712 U.S.B.R. Earth Manual 1963. Reference may also be made to para 3.1.1.1.

3.1.2.2 Instruments for internal vertical movement observations.

A. VERTICAL CROSS-ARM

The system essentially consists of a series of telescoping 1½ and 2-inch sections of pipe with the 1½-inch sections anchored to the embankment by horizontal cross-arms. The installation consists of a base pipe extension, cross-arm units, intermediate sections, and a top extension.

Two variations of the internal vertical movement devices may be placed in earth dams. One installation has a 5-foot vertical cross-arm interval, the other a 10-foot interval.

The equipment required for settlement measurement includes the torpedo, a 5/16-inch wide steel engineer's tape, tape tension handles, tensions spring and reading scale. The elevations of the cross-arm units are determined by lowering the torpedo through the reading scale and into the piping system by means of the steel tape. Pawls on this torpedo successively engage the lower ends of the 1½-inch pipe (measuring points on each cross-arm unit) and upon reaching the bottom of this installations they latch in a closed position to enable the torpedo to be withdrawn from the system. When readings are made, the reading scale is attached to the top section of pipe projecting from the existing installation. The elevations of the levelling plate on top of the reading scale must be established from a permanent benchmark of the dam. Vertical distances from the top of the reading scale to the respective cross-arm measuring points are obtained by using the torpedo and attached tape.

For detailed description relating to construction details location and layout, method of installation, procedure for observations and analysis of observed data etc. refer to Designation E-29 pp. 672-693 U.S.B.R. Earth Manual (1963).

B. SWEDISH WELL

This device which has been successfully used for several dams in Europe, consists of a vertical column

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of large diameter reinforced concrete pipes. The pipes are installed in pits during construction of the embankment and are first separated an inch or two with two wooden wedges or shims. Then as the fill progresses upwards the wedges are knocked out so that each individual pipe section has no connection with the others and is held in place by surrounding embankment material. With this arrangement the pipes are free to move with the embankment and the three components of the movements can be measured.

The Swedish well provided in Rosshaupton Dam (Germany) has reinforced concrete pipe section of 150 cm inside diameter and 95 cm long.

Details of 42 cm diameter Swedish well are shown in Fig. 8.1.9, at page 400 of the publication in Earth and Earth Rock Dams by Sherard, Woodwards and others.

Location and Layout : On account of the open structure of the Swedish well (floating R.C.C. rings) these can be located only either in the core itself or downstream of the core, to ensure prevention of entry of seepage water through the embankment, into these wells.

Installation details have been given in the earlier para of this section.

C. FOUNDATION SETTLEMENT MEASURING DEVICE

The foundation settlement measuring apparatus essentially consists of a base plate placed on the foundation excavation line and a vertical column of steel pipe. The apparatus is comprised of a 2-inch standard steel pipe, a cross-arm measuring unit (base plate) an anchor cross-arm, and an extension of 2-inch pipe to the surface of the embankment.

For details of apparatus location and layout method of installation and observation and analysis of observed data refer to Designation E-31 pp. 704-707 USBR Earth Manual (1963).

3.1.2.3 Instruments for internal horizontal movement observations

The following instruments or apparatus are used for the measurement of horizontal displacement in earthen embankments :

- (i) Horizontal cross-arm.
- (ii) Swedish well.
- (iii) Electrical clinometers.

(i) Horizontal cross-arm

The device consists in embedment of the cross-arm for horizontal movement measurement, the principle on which the device works being similar to that for vertical cross-arm. In these instruments the relative movements between the horizontal cross-arms are

transformed into vertical movement by a linkage cable-counter weight arrangement, and measured by using the torpedo and reading scale as used for vertical cross-arm observations.

Detailed description of this apparatus is given under designation E-31 in U.S.B.R's Earth Manual 1963 (pp. 693-704).

(ii) Swedish well

Refer to para 3.1.2.2 (B).

(iii) Electrical clinometers

Most commonly used instrument is the Wilson slope indicator. Detailed description of the instrument and relevant information regarding procedures for installations, maintenance, observation and analysis of observed data are given in para 3.1.2.4.

3.1.2.4 Wilson slope indicator

(i) Principle and construction

The slope indicator manufactured by the Slope Indicator Co., U.S.A. is a precision instrument for measuring the inclination of a nearly vertical surface. The instrument is lowered down a grooved plastic casing (or aluminium extruded casing) the grooves controlling the orientation of the instrument in a pre-determined direction. Inclinations are taken at frequent interval of depth and are subsequently converted to displacements.

The slope indicator consists of a segment of precision-molded circular resistance element pivoted on instrument ball bearings with an eccentric weight mounted on the rim, enclosed in a watertight aluminium cylinder 2-3/8 inches outside diameter and 15 inches long. Attached to the same inner frame as the pendulum is a relay, which when activated by an electrical current, moves a conductor into contact with the resistance element and subdivides it into two resistances forming one-half of a conventional Wheatstone bridge circuit. The other half, including switches and batteries, is enclosed in a control base at the ground surface. The instrument and control box are connected by a heavy duty cable with yellow marker bands at one foot and red marker bands at five foot intervals. The instrument has two sets of guide wheels. Fixed guide wheels, aligned in a vertical plane parallel to the axis of the instruments, are mounted on one side while on the opposite side are a corresponding pair of spring mounted guide wheels.

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

The electrical supply is from four 6-volt dry cell batteries (RCA No. VS 317 lantern battery or equal) wired in series. These batteries should be checked

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periodically and should be replaced when the voltage drops below 20 volts.

Slope indicator has a sensitivity of one part in 1000 which means that it can detect a tilt of as little as three minutes of arc. This corresponds to a lateral movement of one half inch in 50 feet of depth. Displacements in thin shear zones of less than 1/16 inch can be easily identified.

An accuracy of 1/16" for measurement of movements in thin shear zone should be adequate as this is about 1/6 of the maximum movement in conventional direct shear machine (3/8"). For this purpose, a measurement of 10 minutes of arc is required with a gauge length of 20". To allow for observational and instrumental errors an accuracy of 3 minutes is recommended.

The specially grooved plastic casing is easily installed in 4-inch or larger bore holes. It is available in 5 ft. lengths and is joined together in the field with aluminium couplings.

(ii) Location and layout

Considerations governing location and layout of the inclinometers are as under :—

- (a) When measurement of horizontal strain is necessary where progressive failure is apprehended due to significant difference in the stress-strain characteristics of soils of different zones or different parts of the foundations.
- (b) When it is necessary to ascertain the presence or otherwise of tendency for dangerous internal horizontal movement.
- (c) When problems of long-term stability are involved, an element of uncertainty always remains in the estimation of shear strength. Inclinometers should be installed in locations susceptible to instability on account of the above uncertainty, to enable timely detection of the dangerous movement for remedial measures.
- (d) Inclinometer installations provide very useful indications of tendency for separation between core and the abutment.

Hence it should be provided in

- (1) abutments with steep and irregular profile
- (2) high dams with narrow cores.

(iii) Method of installation

Observations wells consisting of aluminium extruded casing may be provided in the dam as the construction progresses or may be provided in drill holes drilled subsequent to the completion of the embankment.

Procedures for installation are as under :—

A. OBSERVATION WELLS IN EARTH DAMS

I. General Description of Observation Wells

The observation wells consist of 3-inch inside diameter aluminium casing having four continuous longitudinal slots on the inside to guide the Slope Indicator. The casing is extruded in half-round sections, anodized, and then cut into five-foot lengths. Two lengths of half-round sections are joined together in the field to form a full-round section five feet long. Successive lengths of casing are then joined together with self-aligning couplings as the fill is placed. A gap is left at each joint so that as the embankment settles under its own weight the gap is gradually closed. The depth from the top of the casing to each joint is measured at frequent intervals using a special settlement torpedo, and at the same time the inclination of the casing is determined by the Slope Indicator.

II. List of Standard Equipment

- Aluminium casing, half-round, in 5-foot lengths.
- Aluminium couplings, half-round, in 12-inch lengths.
- Wooden plugs.
- Spring clips.

III. Anchor Hole at Bottom of the Observation Well

The bottom section of casing is set into a hole into the foundation of the dam, which is usually rock or competent soil. This pre-bored hole is 4-inches or more in diameter and about three feet deep. Deeper holes are required if it is desired to also measure possible movements of the foundation soils. Bore-hole requirements for deeper holes are given in "Instructions for installation of Observation Wells in Drill Holes" refer para iii (B):

IV. Assembly and Installation of Casing

1. *Assembly of Casing*: (a) Take two lengths of half-round casing and assemble to form a full-round section by inserting the tongue of one section into the groove of the opposite section. Make certain that the two ends are flush with each other.

(b) Snap two spring clips on each side at about one-third points.

2. *Assembly of End Plug*: (a) Assemble the two halves of a 6-inch length of aluminium coupling over the wooden plug at one end and a length of casing at the other end, and snap two spring clips on each side.

(b) Attach the aluminium casing to the wood with several small nails or screws.

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(c) Caulk the six slots at the bottom of the coupling so as to close up the opening between the slot and the wood. This may be accomplished with hemp, cloth, plastic, or any caulking compound.

3. *Installation of Casing* : (a) Lower the first five-foot section of casing to the bottom of the anchor hole, with the wooden plug on the lower end.

(b) Fill the annular space between the casing and the sides of the bore-hole with cement grout and allow to harden, or else fill the space with compacted sand.

(c) Locate the centre of the casing at the elevation of the ground surface by means of a precise survey, both with respect to elevation and horizontal coordinates.

(d) Place and compact fill material around the casing in the shape of a mound, keeping the top of the mound several feet higher than the surrounding fill at all times.

(e) Set another section of casing vertically above the section of casing in the hole, with the slots in line.

(f) Attach the coupling, making certain that all interlocking parts match up properly and that the tongue of one coupling half-section is properly aligned with the groove of the opposite half-section. Snap two spring clips on each longitudinal joint.

NOTE :—Couplings for use in earth dams are furnished in standard lengths of 12 inches. A gap of 6 inches is left between adjacent casing ends, which permits a total compression of ten per cent. Greater or lesser gaps may be left if desired, which may require ordering special coupling lengths.

(g) Repeat steps (d) through (f) until the embankment is completed.

4. *Alternate Installation Procedures* : When the casing is installed, by extending it above the surface of the embankment, as described above, some interference with construction equipment is certain to develop. To minimize this interference, an alternate procedure is to first cap the top of the casing below the elevation of the fill and to then add a 5-foot lift of fill. Next a pit is excavated to uncover the top of the casing, the temporary cap is removed, a 5-foot section of casing installed, and the hole backfilled and tamped by hand. Prior to capping, a set of readings with the settlement torpedo and with the Slope Indicator are obtained.

B. OBSERVATION WELLS IN DRILL HOLES

1. *General Description of Observation Wells*

The observation wells consist of 3-inch inside diameter aluminium casing having four continuous longitudinal slots on the inside to guide the Slope Indicator. The casing is extruded in half-round sections and then cut into five-foot lengths. Two lengths of half-round sections are joined together in the field to form a full-round section five feet long. Successive lengths of casing are then lowered down a pre-bored hole and joined together with special couplings which automatically align the slots. After the casing has

been placed in the drill hole, with one pair of slots for reference in the desired direction, the annular spacing between the casing and the sides of the boring is backfilled with sand, pea-gravel or cement grout, depending on local conditions and availability of materials.

II. *List of Standard Equipment*

Aluminium casing half-round, in 5-foot lengths.

Aluminium couplings, half-round, in 6-inch lengths.

Wooden end plugs.

2 Casing clamps.

Spring clips.

Crimper.

III. *The Drill Hole*

The casing is installed in a pre-bored drill hole 4 inches or more in diameter. The actual diameter of the hole and the method of accomplishing the boring depend upon the type of materials encountered, the depth of the hole, and the equipment available. Best results are usually attained if the hole is between 5 and 6 inches in diameter.

If the ground is firm and the hole will remain open without steel casing, then either a dry hole (drilled with a continuous-flight auger) or a water-filled hole (drilled with a chopping bit or a rotary bit and using water to flush out the cuttings) will be satisfactory. If the hole is likely to squeeze in because of soft soils, or cave in because of sand, a drilling mud may be used and left in the hole upon completion of the boring. In extremely difficult cases a 4-inch ID or larger steel casing may be used and either left in place upon completion of the boring, or removed after the Slope Indicator casing is inserted.

IV. *Assembly and Installation of Casing*

1. *Assembly of Casing* : (a) Take two lengths of half-round casing and assemble to form a full-round section by inserting the tongue of one section into the groove of the opposite section. Make certain that the two ends are flush with each other.

(b) Snap two spring clips on each side at about the one-third points.

(c) With the crimping tool, crimp the external ridge of the longitudinal joint about one inch from each end of the 5-foot section. The purpose of this is to increase the torsional stiffness of the casing by preventing motion along the longitudinal joint.

2. *Assembly of End Plug* : (a) Assemble the two halves of an aluminium coupling over the wooden plug at one end and a length of casing at the other end, and snap two spring clips on each side.

(b) Attach the aluminium casing to the wood with several small nails or screws.

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(c) Caulk the six slots at the bottom of the coupling so as to close up the opening between the slot and the wood. This may be accomplished with hemp, cloth, plastic, or any caulking compound.

3. *Installation of Casing in Drill Hole*: (a) Lower the first five-foot section of casing down the drill hole, with the wooden plug on the lower end, and with the slots properly oriented.

(b) Attach the clamp to the upper end of the casing, approximately 6 to 12 inches from the top.

(c) Set another section of casing vertically above the section of casing in the hole, with the slots in line.

(d) Attach the coupling, making certain that the locking lugs in the casing fit the windows in the coupling and that the tongue of one coupling half-section is properly aligned with the groove of the opposite half-section.

(e) snap two-spring clips on each longitudinal joint.

(f) Attach another clamp near the top of the upper section of casing, remove the lower clamp, and lower the casing down into the drill hole.

(g) If the drill hole is filled with water or drilling mud, to overcome buoyancy, the casing should be filled with suitable fluid.

(h) Repeat steps (c) through (g) until the casing rests on the bottom of the drill hole, then flush casing.

(i) The stick-up portion of casing should be cut off at a suitable height with a hacksaw, preferably at the nearest 1-foot mark to facilitate depth measurements.

NOTE:—The installation is expedited if the casing is pre-assembled and coupled together in units of 10 to 25 feet each. Fifteen-foot lengths are easily installed by hand; longer lengths require a drill rig mast or A-frame.

V. Backfilling

The drill hole should now be backfilled. The materials and procedures used will depend on the particular conditions encountered, and may include any one of the following:

1. *Dry Sand*: This may be used when the drill hole is dry and of small diameter, say 5 to 8 inches. The sand should be clean, dry, and of uniform size: washed concrete sand is acceptable. The sand should be densified by vibrations, either by means of internal vibrator (air-driven and available from the Slope Indicator Company) by means of a concrete vibrator attached to the top of the aluminium casing, or by other suitable means.

NOTE:—If not properly compacted the sand will settle later and produce both vertical and horizontal deformations of the casing which are not the result of ground movements.

2. *Dry Pea-Gravel*: This may be used for a large diameter, dry hole in lieu of dry sand.

3. *Wet Sand or Pea-Gravel*: When the hole is filled with water, clean sand or pea-gravel may be used in similar fashion to a dry hole, except that the sand need not be dry. Compaction by vibration or other means is still required.

4. *Grout*: As an alternate procedure, the annular space between the aluminium casing and the drill hole may be filled with grout. This may consist of a lean cement grout, a clay-cement grout or a thixotropic drilling mud.

In the case of a dry hole, the grout may be dropped around the casing from the ground surface, a small quantity at a time, or it may be injected through a small diameter pipe lowered in the annular space between the casing and the drill hole.

In the case of a wet hole, or one filled with drilling mud, the water or drilling mud must be replaced with grout. This may be accomplished either with an external grout pipe as above, or with an internal pipe connected to check valve through the bottom of the casing.

VI. Alternate Procedures

The procedures described herein apply to the case where the aluminium casing is installed in a prebored drill hole. Where the casing is to be installed in earth dams, or is to be installed in bore-holes through compressible soils which are expected to consolidate, telescoping joints must be used, and provisions must be made to measure the closure of the joints.

VII. Miscellaneous Details

The couplings and the longitudinal joints of the casing are not water-tight; therefore if the bore-hole is filled with drilling mud, the mud may enter the casing as it is installed. Likewise, the injection of grout to fill the annular space around the casing may result in grout entering the casing. In such instances, the aluminium casing should be flushed out with clean water before the grout has hardened. Inflow of mud or grout may be alleviated by taping all exposed joints with waterproofing tape.

The aluminium casing weighs approximately one pound per lineal foot. Thus in dry holes the entire weight of the casing must be carried by the top coupling and by the clamp. Tension across the coupling is carried by compression in the locking lugs transferred from the windows in the couplings. Therefore it is essential that the shoulder of the lugs be square and bear firmly against the window ledges. Rounded lugs may wedge the coupling apart. It is suggested that in deep holes (greater than 100 feet), the number of spring clips be increased on each coupling from four to six or more.

The load of the casing prior to assembly of the new section above ground is carried by friction between the clamp and the casing. The friction depends upon

the tightness of the clamp, and this is adjustable in the field. However, to avoid any possibility of dropping the casing down the hole, the clamp should always be attached below the top locking lug. As an extra precaution, the clamp may be attached below the upper coupling, thus leaving five feet of casing above the ground prior to making the next connection.

iv. Taking Field Measurements

A.—*Preparing Instrument for Use*: 1. Open instrument case and remove instrument.

2. Remove the protective nut from the head of the instrument and the plug from the end of the cable. Connect the control cable to the instrument and tighten the joint with the special wrench provided. This joint must be watertight.

3. Connect the control plug into the control panel receptacle.

B.—*Taking Readings in Observation Wells*.—1. Initially, three readings for each five-foot section of casing are usually taken, i.e., one foot from the top, at the centre and one foot from the bottom of each length of casing. Therefore, if the top section of casing is a complete five-foot section, the initial reading interval will be 1.0', 2.5', 4.0', 6.0', 7.5', 9.0', 11.0' etc. In the event that the top section is less than five feet, the intervals shall be adjusted so that the above described three readings for each five-foot section are obtained. The instrument wheels will not span or be at a joint when readings are taken in this manner. The distance to the instrument as marked on the cable is referenced to the midpoint between the wheels.

2. The instrument is lowered to the desired depth and held stationary. Depths are referenced to the bottom of the red or yellow marker bands. After an interval of approximately five seconds, the control switch is partially depressed to close S-1. A rough adjustment of the galvanometer with the microdial shall be made with the first switch closed. The control switch shall be further depressed to open S-2 for fine balancing of the circuit. The control switch is then released to allow the pendulum to swing free and after approximately three seconds is closed again. The microdial should require no further adjustment for a zero reading on the galvanometer if the pendulum has been at rest, however it is possible that the second reading will vary by a few dial units. Further minor adjustments of the microdial shall be made until the circuit is in balance. The dial units are then recorded

3. The orientation of the readings are governed by the direction of the unsprung pair of slots and at identical depths. The four slots may be conventionally referenced as compass points, namely N, S, E, W. It is further convenient to consider N and E as positive and S and W as negative,

4. A field check for correct dial reading shall be made. The sum of corresponding pairs of readings at identical depths should equal a constant. The sum of the readings for each depth should not vary more than 2 or 3 dial units from their mean. If any sum falls outside this range it indicates either that the two slots are not parallel (due to distortion of the casing, irregular slot depth or dirt in the slot) or else an error in depth, an improper circuit balance, the instrument is not in the slots or possible instrument malfunction. In any case the reading should be checked.

5. For readings below a depth of five feet, the pulley bracket shall be placed on the casing care being taken to remove the bracket on withdrawal of the instrument before the instrument hits the bracket.

6. The instrument should be checked each time it is withdrawn from the casing to ascertain that it is still tracking in the correct grooves. If the instrument consistently fails to track in two or more grooves then it is most probable that a joint has become misaligned. If this occurs it may be possible to make use of a special set of torque rods to reorientate the instrument. These are available from the Slope Indicator Company. They should be consulted as to the best procedure to follow in any particular instance.

C.—*Taking Readings in Observation Wells with Telescoping Joints*: When telescoping joints are used, it is essential that all successive readings be taken at the same relative position in each 5-foot section as the original readings. The best procedure has been found to first survey the casing with the settlement torpedo and thus determine the depth to the bottom of each section of casing. The subsequent slope indicator readings are then taken at these depths less 1, 2.5 and 4 feet.

MAINTENANCE

(a) *Instrument*: All guide wheels should be clean and rotate freely. The spring-loaded guide wheel assembly should be free to pivot. Oil once a day while instrument is in use. Check spring tension and the condition of springs on the spring-loaded guide wheels. If a spring is broken, re-place with one of the springs which will be found in the spare parts can.

Free water inside the case will result in the accumulation of moisture on one or more vital electrical parts. All connections are protected with O-ring seals. However, water may enter if the control cable is not properly connected to the instrument or if the small O-ring is damaged. Hence, be certain that (a) the O-ring is in perfect condition, otherwise replace, and (b) the cable connection is tight.

Water collects on the control cable and on the connection to the Slope Indicator. When the connector is unscrewed, water may run down into the receptacle

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for the cable plug. To minimize this possibility, maintain pressure on the plug by hand as the nut is unscrewed, then wipe the exposed threads and other parts with a dry rag prior to pulling the plug out of the receptacle.

Normally the instrument is not serviced in the field. However, in case of a malfunction, or in the event water accumulates in the case, it may be disassembled by first removing the locking screw and then unscrewing the two halves at the joint. After the threads have been unscrewed, the upper half to which the frame is attached is pulled up out of the case.

In the bottom of the case will be found a small bag of calcium chloride, which may be removed and over-dried to drive off any excess moisture which may have accumulated.

The frame and inner assembly may now be set up right on a work bench or table and the following items systematically checked:

1. Check the pendulum (coil) assembly to verify that there is a little, but not excessive amount of end play in the bearings, and that the pendulum swings freely. When displaced about 30° from the vertical and released, it should oscillate freely for at least ten seconds or more before coming to a position of rest. Adjustment of the bearing holders is possible, but special instructions are required.
2. Check the two spring-loaded contacts at the ends of the axle to assure that they are centered, insulated from the frame, and have sufficient pressure at the points of contact.
3. Check the flap on the relay to assure that the needle bearings are properly adjusted so that there is no end play or wobble.
4. Check the wire contact tip to assure that when the relay flap is depressed, the wire tip is in firm contact with the resistance element, and that the wire is lifted off the plastic shelf attached to the relay flap.
5. Check for general cleanliness and verify that none of the exposed wires are shorting to the frames, etc.
6. Check the position of the wire contact tip so that when the instrument is nearly vertical the contact falls at about the midpoint of the exposed portion of the element.

(b) *Control Box*: The electrical supply is from four 6-volt dry cell batteries (RCA No. VS 317 lantern battery or equal) wired in series. These batteries should be checked periodically and should be replaced when the voltage drops below 20 volts. This may be accomplished without removing the control panel by connecting a voltmeter across No. 2 and No. 3 terminals of the control panel receptacle and depressing the control switches.

When properly wired the galvanometer should rotate in the same direction as the rotation of the potentiometer needle. If the reverse occurs the batteries have been reversed in wiring. This can be corrected by rewiring the batteries.

(c) *Control Cable*: The control cable should be kept clean and free from kinks. It should not be bent over a sharp edge during use. The control panel plug should be kept free from dirt.

If the instrument will still not function properly after all the precautions detailed in sections a, b and c have been observed, it should be returned to the Slope Indicator Company for maintenance and repairs. It is not intended that major repairs be undertaken in the field. Replacement instruments are available.

v. Analysis of Data

The Slope Indicator measures the inclination of the casing in an observation well at frequent intervals of depth. One set of readings is taken in one slot and an additional set is taken in the slot on the opposite wall of the casing. The instrument is adjusted so that the inclination (i) of the casing from the vertical may be computed by the following equation:

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

where

D_N = Dial Reading in North Slot.

D_S = Dial Reading in South Slot.

K = Instrument Constant = 1000 for series 200 instrument

Any change in inclination at a given depth produces a change in dial differences.

$$\Delta \text{Dial} = [D_N - D_S]_1 - [D_N - D_S]_{11}$$

and the change in inclination may be expressed as:

$$\Delta \tan i = \frac{\Delta \text{Dial}}{2k}$$

To compute movements

$$M = (\Delta \tan i) (l)$$

where M = the movement, and

l = distance between successive readings.

If the distance between successive readings is constant, then the total movement (M) of any point with respect to the bottom is given as:

$$M = \Sigma M = \Sigma (\Delta \tan i) (l)$$

$$= \Sigma \frac{\Delta \text{Dial}}{2k} \quad (1)$$

For an instrument constant of 1000, an average distance between readings of 20 inches (3 readings per 5-ft. length).

$$M = \Sigma \frac{\Delta \text{ Dial}}{2000} \quad (20) = 0.01 \Sigma \Delta \text{ Dial}$$

Thus the movements can be computed very easily by summing up from the bottom the changes in dial differences, and multiplying each sum by 0.01, (answer is in inches).

The dial differences taken from the Field Data Sheet are transcribed directly onto the summary sheet. As additional sets are obtained the new dial differences are likewise transcribed onto the summary sheet and the changes are computed algebraically. Starting at the bottom the changes are successively summed up from which the deflections are computed very simply. If the dial changes are positive, the upper portion of the casing is being displaced to the North (or positive slot) with respect to the bottom.

Plots of dial changes and of deflections are prepared.

3.1.2.5 Internal horizontal movement by tilt-meters (clinometers) (telemac or maihak type).

Refer telemac and maihak catalogues and para 3.1.1.9.

3.1.2.6. Internal horizontal and vertical movement by electromagnet probe, and eastman gradient indicator

(Refer Gepatsch Rockfill Dam in Kanner Valley by Lauffer H. and T. Schober of Austria.)

(i) Principle and construction

Polyvinyl chloride tubes which are 2 m in length and 62 or 75 mm in diameter and are connected by movable sleeves are installed upright. The tubes are surrounded by metal plates which are carefully anchored in the fill and regularly spaced 8 m apart. Measurement of the settlement is carried out by electromagnet location of the plates by means of a probe lowered into the tube, while the horizontal movements are recorded with an Eastman Gradient Indicator (ref. page 19 of the reference).

By installing lying tubes similar to the upright tubes, the settlement is determined by the change of gradient of the lying tubes and horizontal deformation is determined by locating the metal plates magnetically. As the tubes are installed with an inclination of 1-2% towards the sloping faces the position of the tubes can also be measured with a hose pipe scale. Fig. 10 of the reference shows a lying tube with the observation chamber and the hose pipe scale as well as towing tackle with guide pulley at the inner end of the tube. Thus the different head pieces of the measuring devices can be brought into any position.

3.1.3. Instruments for Measurement of Displacements in Other Structures.

Based on the principle of working three types of instruments are used for measurement of displacements in structures other than dams. These are (i) Optical (ii) Mechanical and (iii) Electrical.

(i) Optical Instruments

This category comprises instruments such as optical levels, theodolites and Cathetometers. Wild T2 and T3 theodolites and precision level Wild No. 3 are used.

For details relating to displacement measurement by these optical instruments, refer to para 3.1.1.1.

(ii) Mechanical Instruments

This category includes the following most widely used instruments or devices:—

- (a) Invar tapes or rods.
- (b) Dial gauges.
- (c) Microkaters.
- (d) Whittemore-Huggenberger Strain Gauges.
- (e) Liquid level indicators.
- (f) Clinometers and precision levels.

(a) *Invar tape or rods* with dial gauges are used for measuring diametral deformations in tunnels due to rock movements. Use of invar rods with strainmeters (Carlson type or vibrating wire type) and jointmeters depending upon the expected range of movement is described in paper No. 17 and No. 41 in Symposium on the Observations of Structures Transactions Vol. II Lisbon 1955.

(b) *Whittemore Huggenberger Strain Gauges*:— These are used for surface measurement of horizontal and vertical displacements across joints in aqueducts, bridges, spillway piers etc. For details relating to this principle of working method of installations, observations and analysis of observed data, refer to para 3.1.1.4.

(c) *Clinometers*:—For measurement of angular changes clinometers are used. For details on the use of clinometers, refer to para 3.1.1.9.

(iii) Electrical Instruments

For measurements of displacement at points which are inaccessible or are subject to hazards, use of electrical instruments (displacement pick-ups) namely bonded wire strain gauges or unbonded wire gauges of the resistance type or vibrating type may be used. For details regarding these, refer to Chapter 3.3.

3.2. INSTRUMENTS FOR MEASUREMENT OF STRESSES

Stress is generally determined from the observed strain data in various directions obtained from strain gauge rosettes. However, the conversion of strain data to stress in general is quite complicated and requires a knowledge of the elastic constants and creep. As structural design is primarily based on stress rather than strain, the direct measurement of stress in a structure is of great importance.

The instruments most frequently used in dams and other related structures are the following :

- (a) Electrical resistance type
 - (i) Surface Gauge—bonded Wire Dydyc Gauge.
 - (ii) Bonded wire Carlson type.
- (b) Electromechanical or Acoustic type.

3.2.1 Dydyc Gauge

(i) Principle and Construction :

The stress σ_x on any surface is given by the relation

$$\sigma_x = \frac{E}{1-\mu_2} \left(\Sigma_x + \mu \Sigma_y \right)$$

where Σ_x and Σ_y are the strain in the X and Y directions and Σ and μ the elastic constants. The output of a gauge would be proportional to the stress along its axis if a proper proportion of good wire were arranged in the transverse direction. Another factor is the presence of shearing strain which will cause a change in length of the gauge wire. Hence if the strain sensitive wire is placed along one diagonal an equal amount in series with it must be placed along the other diagonal to correct the effect of the shearing strain on the deformation of the wire.

(ii) Number, Location and Layout :

Stress gauges at present manufactured are only suitable for steel structures. These would be suitable mainly for tunnel supports, gates and other steel structures. The number, location and layout will have to be decided on the specific nature of the problem.

(iii) Method of Installation :

Same as for SR-4 strain gauges described under 3.3.4.

(iv) Observations, Forms of Record and Sources of Error :

Observations are directly taken on a direct reading bridge and conversion to stress is made by use of a calibration constant. The major limitation of this commercial gauge stems from its special character. It is produced in very limited range of gauge types and sizes and is consequently too restricted in use.

(v) Analysis of Data :

No special analysis of data is required.

3.2.2 Carlson Stress Meter

(i) Principle and Construction :

The stress meter consists of a mercury filled diaphragm shaped like a plate, with a chamber protruding from one side in which is enclosed a measuring unit. The centre of the plate under the measuring unit chamber is made flexible by introduction of a cavity in the plate which also serves for attachment of the measuring unit chamber. The mercury in contact with this plate deflects it elastically in direct proportion to the intensity of pressure on the diaphragm. The diaphragm in effect acts like a hydraulic multiplier. The measuring unit, which is a small elastic wire strainmeter, measures the intensity of stress as change in resistance ratio of the two coils of the strainmeter and measures the temperature in terms of the series resistance of the two coils in the same way as the strainmeter.

The range of the standard stress meter is 0 to 800 psi. in compression. In this model the sensitivity is about 5 psi. per 0.01 per cent change in the resistance ratio and the series resistance of the two coils changes about 1 ohm for 8°F in temperature. A temperature correction must be made when computing stress indicated by the stress meter. The correction amounts to from 0.5 to 1.5 psi. per degree F change in temperature.

Stress meters are available in two other ranges designed for use in measuring compressive stress in concrete and a companion meter for measuring soil pressures.

The details of stress meter are shown in Fig. II. 3.2-01.

(ii) Number, Location and Layout :

Stress meters can be located only in zones subjected to compressive stress. These are useful to measure the compressive stress distribution or to measure silt loads at upstream face of the dam.

For the measurement of the stress distribution along any horizontal section stress meters may be placed at five locations as recommended for the strain gauge rosettes. The stress gauges may be used to measure vertical stress on horizontal planes, horizontal stress in vertical planes or stress on inclined plane. However due to certain difficulties in installation vertical stress meter alone can be considered reliable. The stress meter location apart from these used for measurement of silt pressure are guided by the consideration given for strainmeters.

(iii) Method of Installation :

Installation.—Since the stress meter measures the force in concrete it is important that there be good contact between the meter and the concrete. In the manufacture of the stress meter, great precautions are taken to assure perfect contact of the mercury and the

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surrounding diaphragm. Similar precautions must be taken in the embedment to make good contact between the meter and the surrounding concrete. A hardened underlying concrete is essential for the placing of a stress meter for measuring vertical or diagonal stress. If placed in fresh concrete, there might be some collection of water at the under surface, and such imperfect contact would make the installation worthless. It is often advisable to place the meters in a recess about 12 inches deep in the top of the previous lift of concrete. The meter for measuring vertical stress would be bedded on to the bottom horizontal surface of the recess, and 45-degree meters (if used) would be bedded onto the 45 degree sloping sides of the recess. The surfaces where the stress meters are placed should be reasonable plain and not too rough. Only if these surfaces are both smooth and plain can the uniformly thin bonding layer of mortar be achieved.

The following steps are recommended as a means to obtain good contact between stress meter and concrete. First wire brush (or otherwise treat) the hardened surface of the underlying concrete to leave a good bonding surface. About 30 minutes before placing the stress meter, mix 80 gms. cement, 120 gms. sand passing the 30 sieve, and enough water to make a plastic consistency. Practice trials should be made to learn just how wet to make this mortar. Just before placing a stress meter, paint the concrete surface with a film of grout made by adding water to a part of the mortar until it has the consistency of cream. This is to dampen and lubricate the surface so that when the stress meter is set down on the mortar pad in the next step, the mortar will not ride over pits or depressions as it squeezes outward, thus leaving voids on the contact surface. After the surface has been painted, about 150 grams of the mortar are placed in the centre of the area where the stress meter is to be seated, and this mortar is shaped roughly into a cone. With a reciprocal rotary motion, the stress meter is pressed down on this mortar cone causing it to squeeze outward and appear around the rim. After the meter has been set in mortar a metal tripod is set on the upper face of the diaphragm, such that weight of about 20 pounds can be placed on top of the tripod a foot or more above the meter. After the meter is well covered with concrete, the weight and tripod can be removed. This practice insures that there is intimate contact between the stress meter and the concrete below. The tripod can be made with slender vertical legs of about 1/4 inch diameter or slightly larger.

Good results in placing stress meters can be assured if preliminary trials are made with a dummy stress meter. A dummy can be made from a steel disc of about one-half inch thickness and about seven-inch diameter. A steel tube or rod can be welded to the centre of one face of the disc to simulate the strain meter unit of the stress meter. A trial consists in bonding this dummy stress meter to a hardened concrete surface according to the directions above. After the bonding mortar has been allowed to harden about 24 hours, the dummy can be removed and the

contact surface examined. The bond should be strong and the appearance of the contact surface should indicate intimate bond with no voids except perhaps a few small air bubbles, which are unimportant here. Although good placement of stress meters is difficult without practice, good placement is easy to achieve after practice.

Stress meters for measuring horizontal stress are placed in the fresh concrete, usually by hand. It is customary to place the fresh concrete around all except a small portion of the upper rim and to tamp the concrete well and allow it to stiffen for some minutes before covering the entire device. Leaving a small portion of the rim exposed as long as practicable helps to insure that the position and alignment are correct. Placement of meters for measuring horizontal stress is usually done at the end of a lift, after the workmen have left, but when the concrete is still plastic. Then, the conductor cables can be carried along the top of the fresh concrete and covered so as to be protected from that time on.

(iv) *Observations, Forms of Record and Sources of Errors:*

Field Measurement.—Readings are taken on stress meters with Carlson testing set or equivalent, according to instruction supplied with the testing set. For determining stress, the resistance ratio is measured with the WHITE GREEN BLACK conductors (in that order). For determining temperature, the meter resistance is measured with the WHITE AND BLACK conductors. If the stress meter has been provided with a 4 conductor cable, the meter resistance can be measured exclusive of the resistance of the conductors, according to instructions supplied with the testing set. If the stress meter is not provided with 4 conductor cable, the resistance of the conductors can be measured indirectly as follows: Subtract the total resistance as measured with the WHITE AND BLACK conductors from the total resistance as measured in two parts with the WHITE AND GREEN conductors, and with the GREEN BLACK conductors, respectively. This difference is equal to the resistance on one pair of conductors, since only one pair was included in the first measurement and two pairs were included in the second measurement. The determination of conductor resistance does not need to be made for every set of reading because it is generally small and its change from time to time is negligible unless the temperature change is large.

Typical form for record is given as Fig. II-3.1.-14. Temperature correction is made as indicated.

The main sources of errors are—

- (a) Defective installation which creates a gap between the meter surface and the concrete.
- (b) Leakage at the cable splices.

These can be obviated by proper care.

(v) *Analysis of Observed Data:*

Computation of stress and temperature.—A typical work sheet for recording measurements and computing temperature and stress is shown in Fig. II-3.2-02. In the heading of this sheet are recorded the exact location of the stress meter and the calibration data. These data which are supplied by the manufacturer of the stress meter are marked with an asterisk, thus.* For determining temperature, the calibration is recorded as "change in temperature per ohm" instead of "ohms per degree" as formerly. This permits the temperature to be computed by a single multiplication, instead of a division, and this is more convenient when using a slide rule. Also, the resistance at 0°F is given instead of at 70°F., as formerly, because this makes the computation simpler.

The calibration constant for stress as supplied by the manufacturer must be revised slightly when long conductor cables are used. The conductor cable introduces an extra resistance which does not change with stress, which consequently makes the stress meter slightly less sensitive.

3.2.3 **Electromechanical or Acoustic Type**(i) *Principle and Construction:*

Electro-acoustic stress meters are designed for the direct measurement of the existing stress conditions.

They are based on a different principle than the strain metric one. With strainmeters the deformation of the measurement base is measured in function of the stress and then we get to the strain value through the elasticity modulus (E). Instead tensiometric capsules belong to the tensiometer or dynamometer class. Such instruments will directly measure the tension state and completely absorb the flux of present forces, without changing the strain distribution on other elements, placed in parallel with the checked one. The simplest measurement is made when the tensiometer is in series connected with the element under examination, and the most difficult one when the instrument works in parallel with others. In the latter case it is necessary that the tensiometer have an elastic deformation equal to that of the other structure elements, in order not to alter strain distributions. Tensiometric capsules are based on deformation measurements in function of the load of two plates, connected to each other, along the circular profile. The measurement is made by means of a vibrating chord, stretched between the two plates. The two plates are different: one is very deformable, while the other can be considered very rigid. The connection between the two plates has been realised so as to obtain a support on a thin zone, at a little distance from the edge. Under the load effect the edge is liable to be deformed in the opposite direction of the present form. It therefore, reduces the side effect of alteration of strain distribution. Several types of tensiometric capsules are made viz. for different materials and loads (concrete, earth etc.). The capsule described hereunder has been designed for stress measurements in big concrete structures.

It consists of a base plate of steel, to which the upper plate is secured. The latter has been designed so as to act as a supported plate and it is stiffly locked to the base plate. Between the centres of the two plates the vibrating chord is stretched to detect deflections in function of the load. The electro-acoustic detecting unit is of normal type: it is suitable for connection to the Galileo electro-acoustic central board.

The coil of the electro-acoustic detector is also used for temperature measurements. A three-pole cable protrudes from the capsule for the connection to the electro-acoustic central board and to the Wheatstone bridge for temperature measurements. The main features of the instrument are as follows:

Diameter—239.8 mm.

Height, h—110 mm.

Weight—about 4 kilos

Area—452 cm²

Measurement range—up to 50 kilos/cm² *i.e.* total load up to 22,000 kilos.

Measurement sensitivity 0.2 kilos/cm² for each graduation (*i.e.* 80 kilos load).

Capsule deflection with 50 kilos load, about 0.04 mm. Also other capsules are made for different load values and of various diameters. The capsule behaviour is almost linear in the whole measurement range. Every capsule undergoes repeated load cycles with the universal material test machine to control its elastic behaviour. The capsule is supplied with a calibration diagram (the electroacoustic central board graduations in function of the load) or with the calibration constant (central board graduations for kg/cm²). The measurements chord is adjusted on the high values of the central board scale (about 460 graduations) so as to grant the whole measurement range. Upper screw plug allows any required chord tension adjustment. Inside of the capsule is placed a silica-gel-drier to avoid condensation effects of the internal air.

(ii) *Number, Location and Layout:*

The same consideration apply as to the Carlson type. The leads from the instrument to the test set could, however, be longer.

Stress capsules are mainly set up near to the foundation ground to study the load distribution in such a zone, also in regard to the lower rocks. Bridge and settling effects can be then, enhanced. The capsules are placed in the central pylon and in the two side ones, transversally interspaced so as to have three or four of them in the section.

Same set-up shall take place at an upper level, occasionally reducing the number in respect to the lower plane. Above described installation foresees the set up of the capsules with their axes in vertical position. In many other cases, when a complete survey

is needed of stresses and strains, some strainmeters shall be set up near to the rosette giving them an orientation, not only with vertical axis, but also with their axes in coincidence with the direction of the strainmeters.

(iii) *Method of installation :*

The capsule is supplied with a rubber sheath around the edge. Such a sheath hinders the concrete from setting on the edge of the capsule itself, letting free, in this way, the upper plates deformations and diminishing the side deviation effect of the present forces. Such a rubber sheath assures also a first stage seal to the external pressure.

The capsule has been designed so as to offer an elastic deformation in function of the load, equal to the concrete one, reckoning an average concrete modulus of 200.000 kilos/cm². Considering though, that when the elastic modulus of concrete differs from the assumed average values, it will happen that, in function of the load, the capsule, although following the concrete in its deformation, will be bound to absorb or to discharge part of the acting strain viz. there is an alteration in the flux distribution of the present strains, with a deviation, either towards the capsule or towards the surrounding mass.

Such an effect may be greatly reduced when, in the setting up operations, attention could be paid to an ideal and limit case, as it would be if the capsule was placed at the basement of a pillar of the same section. This can be realized by making the flux lines to keep their way and to hinder lateral deviations.

To get near to above prescriptions, it is advisable to isolate the concrete superposed on the capsule, from the lateral mass. To this effect, the external edge of the capsule is binded with a thin cardboard, protruding from the upper plane, of about 150 mm. In such a way the strain deviation effect is reduced. We advise to bury the capsule, with the same care adopted for the capsule buried into the structure, in a concrete block of equal composition than that used in the structure (dimensions of the block, at least, 540×540 × 700 mm) and to submit same to different loads by means of the universal material test machine. The capsule behaviour shall then be in parallel with the concrete mass.

While designing our capsules, many tests have been made with these test blocks and found that strain deviation effect is reduced to about 10% when an insulating cylindrical baffle is used, at least 110—120 mm high.

When installing the instrument, take care that the capsule leans with the bottom on a very smooth and stout surface and that the concrete is then tamped against the upper plane of the capsule, to assure good adhesion and continuity.

It has to be kept in mind that capsule deformations in function of the load are very small (a few hundredths of mm.) and therefore the connection between the surfaces which transmit the strains must be sure and continuous.

For the set up of the capsules, the following prescriptions must be followed :

1. Don't let the capsule open, to avoid drier exhaustion.
2. Make sure that the chord tension corresponds to the high central board scale values (460-470 graduations).
3. Close carefully the upper plug (by means of red-lead putty and hemp on the threading, sparingly used).
4. Connect the leads by means of the standard metal outlet fitting which has to be screwed on the capsule (see chapter regarding connections).
5. Lean the capsule on the supporting plane. Apply a thin layer of cement mortar to connect the support plane with the lower part of the capsule, which has been previously binded with the cardboard baffle, which should protrude of 120-150 mm. from the upper plane.
6. Complete the casting around and above the capsule and tamp strongly the concrete. A careful vibration of the concrete should then follow with the utmost care.
7. During the installation, check often the capsule operation with the electroacoustic control board.

(iv) *Observations, Forms of Record and Sources of Error :*

The observations are taken by means of a strainmeter control board.

In practical applications the length of the chord varies according to the required amplifications, from $l = 200$ mm. to $l = 80$ mm. The diameter of the special high resistance steel wire varies from 0.4 mm. to 0.3 mm.

The wire should be free from abnormal drawing and mounting tensions.

The frequency range varies from 350—400 Hz to 900 Hz.

STRAINMETRIC CONTROL BOARD

In the strainmetric control board the fundamental principles, above mentioned, are practically realized as follows :

Fig. II-3.2-03 represents the operating scheme of the strain-metric control board. The strain-meter chord and that of the control board are excited by an

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electromagnetic impulse. Electromotive forces are therefore generated of equal frequency than that of the vibrating chord. A thermoionic amplifier generates an amplification of the currents of the microphonic circuit and feeds a loudspeaker.

The detection or evaluation of unisonance condition between the two chords (standard and measurement) is obtained either acoustically by means of the loudspeaker or optically by viewing the beats of a syntony indicator (electronic tube 6E5-GT).

In this tube is observed a dark angular sector against the fluorescent bottom such dark angular sector appears with a varying amplitude and oscillating when far from syntony. The visible oscillations have the same frequency as the present beats.

When syntony is reached, the amplitude of the dark angular sector increases regularly, *i.e.* without oscillations.

The optical detection does not increase the sensitivity, but it makes easier the measurements, especially in presence of noises.

The push-button selector switch insets a milliammeter in the strainmeter circuit, so that the electric continuity of the circuit may be checked. On the instrument dial are shown in red the normal resistance values.

The push-button selector switches (E), (C) and (E+C) will excite respectively, the chords of the strainmeter, of the control board or both. While performing the measurements, the chords of the strainmeter and those of the control board are alternatively excited by means of short pulses, and the tension of the latter chord is adjusted by means of button (L) until syntony is reached, annulling the beats.

A dial, fixed to the standard chord control, is divided in 500 parts.

In the measurement it is possible to appreciate the beat annulment with an accuracy of 1 graduation.

The control board frequency varies from 300 Hz. to 900 Hz. but another range of frequency may be selected, by adjusting the calibration button of the standard chord.

The difference between the two dial readings, made before and after the strainmeter displacement, valued with its own sign and multiplied by the strainmeter constant, gives outright the value of the measured strain.

In the control board is enclosed an INVAR electro-magnetic tuning fork, the frequency of which is practically independent from temperature variations.

The tuning fork has its own vibration frequency of 500 Hz.: it is inserted with the strainmeter selector like any measurement strainmeter.

The standard chord of the control board is calibrated by bringing its vibration to syntonize with the tuning fork one, for a preselected number of dial reading (100 graduations). The operator is therefore enabled to ascertain whether the initial calibration has remained unchanged during a series of measurements or at a distance of time.

The readings of the standard chord are independent from temperature variations, because the chord dilatation coefficient and of its support are equal.

Twelve measuring instruments may be directly connected to the input terminals of the control board. With an independent selection panel it is possible to increase the number of the controlled instruments. The electric connection between control board and strainmeters will be realised by means of a bipolar cable of equal section of lead 's' 1 mm². Distance between control board and strainmeters can reach 500 meters.

When selecting and setting up the connecting cable, following elements must be kept in mind:

1. The strainmeter circuit is part of the grid circuit of the amplifier (care must be then taken to reduce the circuit self-induction and mutual induction in regard to other nearby circuits—A.C. mains, etc.).

2. The electric resistance of the strainmeter is about 30 ohms and the intensity of current of exciting pulse is around 0.2—0.25 Amperes. When connecting strainmeter to the central board, the right polarity has to be watched in order to give a concordant pulse attraction to the permanent magnet, mounted on the strainmeter.

The control board is housed in a wooden cabinet. Overall dimensions are: 400 × 330 × 240 mm. weight 18 Kgs. The control board measurement range allows measurements with any 'GALILEO' strainmeter and electroacoustic instrument.

PARTICULAR INSTRUCTIONS ON THE STRAINMETRIC CONTROL BOARD

The control board is fed with A.C. single phase voltage (45—50 Hz.). On the panel is mounted the selector switch of the feeding voltage. It is possible to select among the following voltages: 110—125—140—160—220 Volt. Power input: 80 Watt.

If the mains are supplying direct current, or if it is necessary to use batteries, then a little rotary converter should be used or a vibrator with a step-up transformer.

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The instruments most frequently used in dams and related structures are:

- (a) Mechanical Gauges
 - (i) Whittemore-Huggenberger detachable gauges.
 - (ii) Johnsons twisted strip gauges.
- (b) Optical Gauges — Huggenberger Gauges
- (c) Electrical Gauges —
 - (i) SR — 4 Bonded wire gauges
 - (ii) Carlson Type Unbonded wire gauges
- (d) Electro-mechanical — Vibrating Wire Type or Acoustic Gauges.

3.3.1 Whittemore—Huggenberger Gauges

(i) Principle and Construction

The basic element is a sensitive (10³ mm) dial gauge attached between two frame members. These members are provided with 45° conical central points which are kept at fixed distance apart by two elastic hinges. Any movement between the two pins is recorded by the sensitive dial gauge.

On the surface of the structure where strains are required, studs or inserts are fixed which have conical depressions at the centre. During fixing the inserts their distance apart is exactly defined by separating these by a standard bar with pins at fixed distance.

For further details regarding inserts see Section 3.1.

(ii) Number, Location and Layout

These gauges can be used to measure strains only at accessible locations hence their application in the dam as strain measuring device is restricted. Measurement along the surface of the galleries have been made by these instruments. These have also been used to measure the stresses on the downstream face of the dam. The greatest utility of this type of gauge is in measurements on other concrete structures. The number, location and layout would depend on the specific nature of each particular investigation.

(iii) Method of Installation

Same as under 3.1(iii).

(iv) Observation, Forms of Record and Sources of Error

See 3.1 (iii).

(v) Analysis of Data

See 3.1.

In the analysis of strains in general the strains in two perpendicular directions and also at 45° to one of these is required. The principal strains could then be calculated from these by the theory of elasticity (see CWPRS-ARM—1964).

3.3.2. Johnsons Twisted Strip Gauges—Mikrokater

(i) Principle and Construction

The chief element of these instruments is a dial gauge where the dial indicates the rotation of a rectangular twisted strip made from a special alloy. Very small extensions of the twisted strip give large rotation which are indicated by the dial. No mechanical gear or lever magnification is necessary and hence the problem of backlash is eliminated. The sensitivity can be as great as 10⁻⁴ mm. The gauge length again depends on the type of instruments and ranges from 5 mm to 10 cms.

(ii) Number, Location and Layout

These gauges although demountable cannot be removed during a set of observations. These are delicate and are suitable for short term observations. These are highly suitable for observations on concrete and prestressed structures while being loaded and unloaded. Due to their extremely small gauge length they are specially suitable for stress concentration measurements in steel parts such as gate and valves and structural members. The number, location and layout will have to be assessed on the basis of a specific investigation.

(iii) Method of Installation

The instrument is placed on the surface and tied to it with springs. The delicate gauge is loaded normally but while taking reading it is unloaded.

(iv) Observation, Forms of Record and Sources of Errors:

The records are kept of the readings after loading and unloading. The interval of time must be made the same after each loading and unloading to correct for creep effect.

The chief source of error is in attaching the gauge firmly to the structure. If this is not achieved erratic results ensue.

(v) Analysis of Data...

Same as for other surface gauges.

3.3.3 Optical Huggenberger Gauges

(i) Principle and Construction

This gauge has been extensively used in the Continent for measurement of strain in structures and their models. The magnification of the displacement between two knife edges is achieved by means of a compound lever and optical level. The accuracy can be 10⁻³ mm. The general details of the gauge are given in Fig. II-3.3-01.

(ii) Number, Location and Layout

This again is a surface gauge and have to be attached on the surface. This is mainly useful to concrete and other structure for short time tests in which

quick loading and unloading is possible. The number, location and layout again depend on the nature of investigations. These gauges must be used in form of a rosette of at least three gauges for complete analysis of strains.

(iii) *Method of Installations*

The technique of installation is similar to that of the Johnson gauge and the gauges are attached by means of springs and special fixtures.

(iv) *Observations, Forms of Records and Sources of Errors*

Same as for Johnson gauges.

(v) *Analysis of Data*

Same as for other surface strain gauges.

3.3.4 **SR-4 Bonded Wire Gauges**

(i) *Principle and Construction*

The SR=4 named after the inventors Simmen and Runge are very versatile and sensitive gauges. These depend on the change in the electrical resistance of a metallic wire with stress. The percentage change in resistance divided by the corresponding percentage change in length *i.e.*, strain is the gauge factor K_s .

$$K_s = \frac{dR/R}{dL/L}$$

The choice of the wire to be used is primarily based on the gauge factor K_s . However, the change in resistance with temperature is also an important factor. The most commonly used gauge materials are Adrance, Iso Elastic and Nicrome. The actual compositions of these alloys are approximately as follows:

Adrance : 54 — 55 % Copper
44 — 46 % Nickel K_s 2.1

Iso Elastic : 36 % Nickel
8 % Chromium
4 % Manganese, Silica and Molybdenum K_s 3.6

Remainder — Iron, Carbon, Vanadium

Nicrome : 75 % Nickel
12 % Iron
11 % Chromium K_s 2.0
2 % Manganese.

The yield point of the wire should be high compared to that of the base member to which the gauge is attached. This is always the case with the above materials. Other alloys such as chromel, manganin, phosphor bronze and constantan are also used for special gauges.

Filament material is the major variable. Since the quantity actually measured is changed in resistance, it is obviously desirable that the ratio of resistance change to strain change be as high as possible. This ratio (ohms per ohm divided by inches per inch) is known as the gauge factor. If this were due entirely to dimensional change, we would expect, from Poisson's Ratio, that the gauge factor of any wire would be approximately 1.6. It is found, however, that materials vary widely in this respect, that some, in fact, actually have a negative gauge factor.

The three materials which are used in most gauges are a copper-nickel alloy (Adrance) which has a gauge factor of approximately 2.0, a nickel-chrome alloy (Nichrome V*) with a gauge factor of 2.2 and an alloy of the Iso-elastic type (36% Ni; 8% chrome; 0.5% Mo; Bal Fe) having a gauge factor of 3.5. The use of these three different filament materials is an example of the compromises that are so frequently necessary in engineering.

Strain gages are somewhat temperature-sensitive as well as strain sensitive. When strains are to be held steady over a period of time, then changes to another strain level for another period accompanied by ambient temperature changes (an incremental loading of an airplane wing or other structure, for example), it is desirable to use a gage which has inherently low temperature sensitivity, even if this is accompanied by lower strain sensitivity. For such applications, use of the copper-nickel filament is indicated. SR-4 gages are designated by the prefix A, as A-1, AB-1, etc. for wire gages, and by the prefix FA as FA -50-12, FAB-50-12, etc. for foil gages. For measurement of strain at elevated temperatures above 500°F, gages manufactured of nickel-chrome alloys are used. These gages made in wire construction are designated HT types 200 and 800. Gages manufactured in foil configurations are designated by the prefix FNH. Some platinum alloys are employed for high temperature strain measurements such as the wire HT Type 1200.

For investigations of rapidly varying strains, where the test period is short and there is little or no temperature change, it is possible to take advantage of the greater output and fatigue strength of the Iso elastic type of gage, despite its greater temperature sensitivity. These gages are designated by the prefix C as C-1, CB-1, etc.

BLH gage filament construction is determined by several factors. It is necessary that the resistance of the gage be held at some convenient figure and early in the development of strain gauge 120 ohms was selected as a convenient value. It was not difficult to make flat grid gages (continuous wire in a single flat plane) of this resistance when gage lengths were in the neighbourhood of 1 inch. Users of the SR-4 gages, however, quickly began to demand shorter and shorter gage lengths. Production problems involved in such gages made it necessary to abandon the flat grid type and adopt the helical grid or wrap-around construc-

tion (continuous wire in two planes). Gage lengths were carried progressively downward through one-half inch, one quarter inch and finally down to as little as one-sixteenth of an inch. Later advances in fine wire technology have made it possible to manufacture short grid gages in the flat grid type and they should be used in preference to the wrap-around gages.

For convenience in measuring strains in different directions gages with multiple grids were developed and they may be obtained in both the wire and foil configurations. The three and four arm "rosette" gages are the most widely used to resolve the principle strain directions and magnitudes. The 45° configurations are particularly applicable to Mohr's Circle solutions.

Performance of the etched foil gage is sometimes superior to an equivalent wire gage for static strain measurements because they exhibit negligible hysteresis and zero shift. The strain sensitivity of foil gages is from five to eight per cent higher than equivalent wire gages and they have an inherently low transverse sensitivity which is desirable in the resolution of bi-axial strains.

Several types of base or carrier materials are used to support the filament wire or foil grids. For most room temperature applications, this carrier is a nitrocellulose impregnated paper which provides a very sturdy and satisfactory gage. In many strain gauge applications, however, ease of application and fast drying time are factors. Hence, a base was developed which uses a special thin paper. This type of carrier permits quicker evaporation of the nitrocellulose cement solvent it also makes for remarkably easy and fast installation on flat or curved surfaces. All paper type foil gauges and some wire gauges employ the thin paper construction. Most of the paper based gauges can be used in the temperature range—320°F to +180°F. Examples: Wire gauges: A-3, A-5-1, AR-2. Foil gauges: FAP-50-12, FAP-25-12.

For slightly higher temperature applications and the ultimate in gauge flexibility, an epoxy backing is supplied on some foil gauges. This type of gauge can be used in the 320°F to +250°F range. Epoxy backed foil gauges are designated FA-25-12, FA-12-12, etc.

Still higher temperature considerations led to the development of another type of base. Phenolic (Bakelite) impregnated cellulose or glass fibres are used with the result that the strain sensing filament is firmly embedded in a phenolic wafer. The Bakelite type wire gauges will stand continuous operation at 350°F and will give limited service up to 500°F. The Bakelite type foil gauges will stand continuous operation at 450°F and will give limited service up to 600°F. For long term strain measurement at temperatures above 450°F, it becomes necessary to utilize ceramic cements to obtain the bonding agent stability necessary for strain gauge operation.

Adrance foilgauge provided on a strippable vinyl backing which allows the filament to be set directly into ceramic bonding cement are identified by an 'H' in their designation such as FAH—25-12, etc. Some foil gauges are available in a free bonding state for ease of application (no backing) and are identified by an 'O' such as FAO-50-12, etc.

Dynamic strain measurements to 1400°F and static strain measurements to 700°F can be made by using Nichrome V foil gauges with a strippable backing. They are designated FNH-25-12 etc. Nichrome V foil filaments are also available in the free bonding state (no backing) for these high temperature environments and they are identified by an 'O' in their designation such as FNO-50-12, etc. Wire, free filament gauges are available in a variety of strain sensing materials and with a thermocouple included if desired for correction of temperature effects. These gauges have an HT in their designation such as HT-200—2A, etc.

The cements used in attaching the filament to the base carrier and to the test specimen also vary. Nitrocellulose, solvent release type cements, such as Duco or the fast drying BLH SR-4 cement, are used with paper base gauges only and are serviceable to about 180°F. The SR-4 cement is primarily used with the thin paper gauge types. Bakelite type gauges selected for their inherent dimensional stability and long life, but where a room temperature cure is needed and a service temperature of not more than 150°F is expected, may be bonded with Eastman 910 cyanoacrylate cement or EPY-150 epoxy cement. For static work up to 400°F and for rapidly varying strains at temperatures to 600°F, we suggest the use of phenolic resin (Bakelite type) gauges together with Bakelite type cement or more preferably EPY-400 epoxy cement. For strain measurements above 600°F AL-P1 cement or PBX cement is required. All of the cements mentioned are available with application tools and bonding accessories in kit form. The contents of the kits are given in the BLH Strain Gauge Price List.

Still another variable in the construction of strain gauges is the type of connection between lead wire and the filament of the gage. The simple soldered junction between the fine filament and heavier lead wire or ribbon originally used is entirely satisfactory for ordinary stress analysis under static or slowly varying loads. In these applications, each gage is subjected to a relatively small number of stress changes of moderate magnitude. When, however, gauges are to be used in fatigue study, fatigue failure of the filament wire at the soldered joint is sometimes experienced. Similarly, when large strains are involved, hysteresis becomes noticeable and premature failure may occur. The dual lead wire strain gauge was developed to meet this situation, effectively increasing fatigue life and markedly decreasing hysteresis at high strains. Dual lead gauges are designated by the letter 'D' in the type designation. Examples: AD-1, ABD-7, CBD-1, etc.

REPORT OF THE COMMITTEE ON INSTRUMENTATION FOR DAMS AND HYDRAULIC STRUCTURES

STRAIN GAUGE TEMPERATURE COMPENSATION

In addition to the five basic variables previously mentioned, compensation for temperature effects on strain measurements must be considered when variable temperatures are to be encountered during any particular tests or when strain readings must be referenced to a no load ambient temperature condition.

The "Active-dummy" type of compensation was the original method used to automatically correct strain readings for temperature effects. Although the oldest in concept and use, this method still offers many advantages that cannot be matched by newer methods.

The principle of the "Active-dummy" method of temperature compensation is quite basic. In its simplest practice an "Active" gage is bonded to the test specimen and a second identical gage ("DUMMY") is bonded to a separate tab or small block of the same material as that of the test specimen. The "dummy" gage is then physically arranged so that it is exposed to the same temperature environment as the "active" gage but isolated from mechanical strain. In the unstressed state, both gages will then have the same response to temperature changes. The "active" and "dummy" gages are connected into adjacent arms of the measuring (Wheatstone) bridge.

In this configuration the "dummy" gage cancels the effect of temperature changes upon the resistance of the "active" gage and the bridge balance will be responsive only to mechanical strain imposed on the active gage.

Where the "active-dummy" compensation method cannot be applied three types of self-temperature compensating gages are available.

Selected Melt:—(Wire and Foil) Selected melt gages are made of constant or 800 alloy with a selected or adjusted temperature coefficient for minimum response to temperature changes as bonded to various materials. These gage types are designated by the letter "S" followed by the numbers 6, 9, or 13 representing the approximate thermal coefficients of expansion of ordinary steels, some stainless steels and aluminium respectively. The temperature coefficient tolerance for this gage type is maintained at +1% over the temperature range of +50°F to +250°F for Constantan, and 75°F to 600°F for 800 alloy.

Dual Element:—(EB and EBF Type Wire). The EBF type wire gages have two constant wire elements connected in series. One element has a positive temperature coefficient and the other a negative coefficient which are combined in the resistance ratio to produce compensation on a particular material over two specific temperature ranges. Gages marked "—" (minus) are designed for the —50°F to +300°F range and those marked "+" (plus) for +50°F to +300°F temperature range. EBF gages are made primarily for mild annealed steel (designated "S") quartz ("Q"),

Titanium ("T" fits closely on A 70, C 130-AM, C 120 AV and T 1104) and 2024-T4 duraluminum ("D"). Their use on other materials may result in less effective temperature compensation because of variation in thermal expansion characteristics to be found among the various metals, even those of the same general class. Temperature compensated gages for other materials can be produced on special order.

The temperature coefficient tolerance for this gage type over both temperature ranges is maintained at $\pm 1/4\%$.

Dual Element: Universally compensated Foil:— Employs a Nichrome V strain sensing element combined in half bridge form with a C.P. Platinum temperature sensing element. Temperature response characteristics can be adjusted by varying the parameters of a simple external circuit so that a single gage type is capable of temperature compensation on a wide variety of materials at a specific temperature or over any temperature range. The Bakelite base gage, FNB-50-12-E, may be used from —400 to +500°F and type FNO-50-12E for use in ceramic cements to +850°F. This gage type is capable of temperature coefficient tolerances of $\pm 1/10\%$ over very broad temperature ranges.

3.3.5 Special Purpose Strain Gages

(i) To complement the standard line of gages, a variety of special purpose types are available. They are:—

1. Spiral foil gages designed to measure tangential strain in diaphragms. Example: FAS-25-12.

2. Weldable foil gages which have been pre-bonded and temperature cycled on stainless steel shim for maximum stability and can be applied by spot welding to the test piece. Example: FABW-25-12.

3. A wire stress gage can be used to measure stress directly, on steel, example CV-28-2.

4. The foil stress-strain gage has two elements at 90° to each other with the resistance ratio set for a particular Poisson's Ratio so that stress along the gage sensing axis or strain in either direction can be calculated directly from the resultant output. Example: FAB-28-S6.

5. Types to measure strain in concrete designated Valore gages are encapsulated in a waterproof brass container to protect the filament when it is installed in the concrete when pouring. Example: AS-9 or self temperature compensated ES-9. Some gages of conventional design but with long gage length (i.e. FA-600-75 and A-9 series) are used for surface strain measurements on concrete.

6. The post yield gages can be used to measure strains upto 10% for the PA-3 (100,000 micro inches per inch).

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Shorter gage lengths are provided by the PA-5, PA-7, and PA-8 which limit the strain range somewhat. PAR-7, 3 element 45° post yield rosettes and monofilament gages are also available.

(ii) Number, Location and Layout :

Most of the SR-4 gauges are suitable for strain measurements on the surface. The gauges are available in the form of rosettes which consist of three gauges in three directions mounted together. The actual number of gauges depends on the nature of the problem for the measurement of strains in steel supports in a tunnel for example single gauges may be posted along the length of the support 8 or 10 locations to give the form of bending and load. For measurement in concrete, longer gauges will be required and location and layout will again depend on the specific nature of the problem. These gauges may also be applied in certain investigations directly on the rock surfaces.

(iii) Method of Installation :

From the standpoint of installation there are several different types of strain gauges but the general instructions apply to all.

1. Paper base wire and foil gauges.
2. Epoxy base foil gauges.
3. Bakelite type wire and foil gauges.
4. Strippable and free handling foil and wire gauges.

Preparing the Surface : The surface must be even, but preferably not too smooth or highly polished. All loose scale and rust must be removed from metal surfaces, and, if the surface is very smooth or polished, it is desirable to roughen it with about 180 grit emery paper.

A light sandblast gives the ideal surface for bonding either paper or Bakelite gauges to a metal, although it is not necessary except under unusual conditions. Certain metals notably copper, brass, beryllium copper, aluminium, magnesium and titanium bond very well when sandblasted or well roughened but tend to bond indifferently if the surface is very smooth or oxides are not removed just prior to gage application.

Cleaning : The surface where the gage is to be installed must be meticulously cleaned. Scrub with a clean solvent such as acetone or trichlorethylene using a clean cotton swab or rag. If wax or the residue from petroleum products is present, clean with toluol first, then follow with the acetone or trichlorethylene.

The surface is clean enough for gage installation when a solvent saturated fresh white swab rubbed briskly over it shows no trace of colour or dirt. Extreme cleanliness is essential if the gage is to be applied to

a very smooth or highly polished surface which has not been previously roughened. The surface must be kept clean until the gage is installed; care must be taken to avoid even touching the surface with the fingers.

Cementing : It is important to use a liberal amount of cement and to avoid forming air pockets in the cement. Always let the weight or spring clamping pressure squeeze out any excess cement. Do not try to make the application look too neat by squeezing out the excess cement before applying weight or a clamp. A properly installed gage will always show excess cement that has oozed out entirely around its periphery. On the other hand, a too liberal coating of cement will create a thick glue line and result in possible measurement errors.

Clamping : When clamping is necessary, the use of C-clamps is not advised. They tend to apply too much pressure at first, while the cement is soft and too little later, while the cement is hardening and needs the pressure. Use a weight, a light spring clamp or a spring loaded clamp depending upon the amount of pressure required.

(iv) Observations, Forms of Record and Sources of Error :

The observation of strain is made on strain indicators which consist of a D.C. or A.C. bridge and suitable amplifiers. There is arrangement for gage factor adjustment. The readings are obtained directly in micrometers per inch. The forms of observation should give the initial and final reading after each cycle of loading and unloading.

The principle source of error is the instability of the gage.

One of the principal causes of gage instability is moisture which is absorbed into the body of the gage. The effects of moisture are (a) dimensional changes of the bonding medium and cement which cause resistance change due to the resulting strains set up in the gage filament and (b) changes of resistance due to the conductivity and corrosion of the strain sensing materials caused by absorbed moisture. A secondary cause of instability is the long time "aging" of the bonding medium and cement. The latter is largely corrected by through "post curing" at recommended temperature (accelerated aging). Both of these effects are minimized by adequate moisture-proofing properly applied.

Experience has shown that moisture proofing is usually unnecessary under most conditions and that a large percentage of SR-4 gauges used are given no moisture proofing or other protection, and yet they give satisfactory service. It is the exception rather than the rule which is the subject of these notes.

As has been pointed out BLH Strain Gauges are of several types. The paper and cement gauges are more sensitive to aging and to changes in humidity

than are the Bakelite wire or foil gages, and hence they do not possess equivalent inherent stability. Clearly, then the Bekelite wire and more particularly the foil type of gage (properly moisture-proofed) is indicated where high stability over a long period of time is essential.

Other common causes of instability are (a) improper installation, (b) insufficient drying or curing of the cement, (c) damaged or greatly overstressed gage, (d) imperfect connections, (e) use of frayed or worn-out stranded lead wire, (f) "leakage resistance" in switches, insulation etc. Such causes are matters of technique and, while of importance, are not treated here.

(v) Analysis of Data :

The main consideration in the analysis of strains obtained from individual gauge of rosette is computation of the principal stresses on the surface. The analysis will be illustrated by the case in rectangular The strains measured by the three

gages are ϵ_1 , ϵ_2 and ϵ_3 and the angle there made with the axis or the direction of ϵ_1 are : $\phi_1 = 0$, $\phi_2 : 45^\circ$ and $\phi_3 : 90^\circ$.

From the well known relation—

$$\epsilon_{\phi} = \frac{\epsilon_x + \epsilon_y}{2} + \frac{\epsilon_x - \epsilon_y}{2} \cos 2\phi + \frac{\gamma_{xy}}{2} \sin 2\phi$$

We obtain

$$\epsilon_1 = \frac{\epsilon_x + \epsilon_y}{2} + \frac{\epsilon_x - \epsilon_y}{2} = \epsilon_x$$

$$\epsilon_2 = \frac{\epsilon_x + \epsilon_y}{2} + \frac{\gamma_{xy}}{2}$$

$$\epsilon_3 = \frac{\epsilon_x + \epsilon_y}{2} - \frac{\epsilon_x - \epsilon_y}{2} = \epsilon_y$$

Solving for ϵ_x , ϵ_y and $\epsilon_{\gamma_{xy}}$

$$\epsilon_x = \epsilon_1$$

$$\epsilon_y = \epsilon_3$$

$$\gamma_{xy} = 2\epsilon_2 - (\epsilon_1 + \epsilon_3)$$

The maximum direct strains and shearing strains are given by

$$\epsilon \text{ max.} = \frac{1}{2} (\epsilon_x + \epsilon_y) + \frac{1}{2} \sqrt{(\epsilon_x - \epsilon_y)^2 + \gamma_{xy}^2}$$

$$\epsilon \text{ min.} = \frac{1}{2} (\epsilon_x - \epsilon_y) + \frac{1}{2} \sqrt{(\epsilon_x - \epsilon_y)^2 + \gamma_{xy}^2}$$

$$\gamma \text{ max.} = \frac{1}{2} \sqrt{(\epsilon_x - \epsilon_y)^2 + \gamma_{xy}^2}$$

$$\phi_p = \frac{1}{2} \tan^{-1} \frac{\gamma_{xy}}{\epsilon_x - \epsilon_y}$$

where ϕ_p is the angle between the direction of the maximum principal strain and the axis X. Substituting value of ϵ_x , ϵ_y and γ_{xy}

$$\epsilon_{\text{max.}} = \frac{1}{2} (\epsilon_1 + \epsilon_3) + \frac{1}{2} \sqrt{(\epsilon_1 - \epsilon_3)^2 + [2\epsilon_2 - (\epsilon_1 + \epsilon_3)]^2}$$

$$\epsilon_{\text{min.}} = \frac{1}{2} (\epsilon_1 + \epsilon_3) - \frac{1}{2} \sqrt{(\epsilon_1 - \epsilon_3)^2 + [2\epsilon_2 - (\epsilon_1 + \epsilon_3)]^2}$$

$$\gamma_{\text{max}} = \frac{1}{2} \sqrt{(\epsilon_1 - \epsilon_3)^2 + [2\epsilon_2 - (\epsilon_1 + \epsilon_3)]^2}$$

$$\phi_p = \frac{1}{2} \tan^{-1} \frac{2\epsilon_2 - (\epsilon_1 + \epsilon_3)}{\epsilon_1 - \epsilon_3}$$

Using the relationships

$$\sigma_{\text{max.}} = \frac{E}{1 - \mu^2} (\epsilon_{\text{max.}} + \mu \epsilon_{\text{min.}})$$

$$\sigma_{\text{min.}} = \frac{E}{1 - \mu^2} (\epsilon_{\text{min.}} + \mu \epsilon_{\text{max.}})$$

$$\tau_{\text{max.}} = \frac{E}{2(1 - \mu)} \gamma_{\text{max.}}$$

we obtain the principal stresses and the maximum shearing stress

$$\sigma_{\max.} = \frac{E}{2} \left[\frac{\epsilon_1 + \epsilon_3}{1 - \mu} + \frac{1}{1 + \mu} \sqrt{(\epsilon_1 - \epsilon_3)^2 + [2\epsilon_2 - (\epsilon_1 + \epsilon_3)]^2} \right]$$

$$\sigma_{\min.} = \frac{E}{2} \left[\frac{\epsilon_1 + \epsilon_3}{1 - \mu} - \frac{1}{1 + \mu} \sqrt{(\epsilon_1 - \epsilon_3)^2 + [2\epsilon_2 - (\epsilon_1 + \epsilon_3)]^2} \right]$$

$$\tau_{\max.} = \frac{E}{2(1 + \mu)} \sqrt{(\epsilon_1 - \epsilon_3)^2 + [2\epsilon_2 - (\epsilon_1 + \epsilon_3)]^2}$$

Similar equation can be derived for other types of rosettes. Nomograms are available for rapid solutions.

rise in temperature or subtracted for a decrease in temperature.

3.3.6 Carlson Strainmeter

(i) Principle and Construction:—(Fig.II-3.3-02)

The Carlson strainmeter depends on the principle that the electrical resistance of a wire changes with the strain. In contrast with the SR-4 type gauges only the ends of the wire are attached to the structure under consideration.

The Carlson strainmeter is in the form of a long cylinder with flanges on each end to anchor the ends of the meter to the surrounding concrete. Within a flexible brass cover is a steel framework on which are supported porcelain spools around which are wound under 100,000 psi tension, two equal coils of fine steel music wire 0.0025 inch in dia. When the ends of the meter are pulled apart by expansion the outer or the expansion coil elongates and increases in tension and resistance. At the same time the inner or contraction coil decreases in tension and resistance as it shortens. Changes in the ratio of the resistance of the outer (expansion) coil to the inner (contraction) coil is used as a sensitive measure of length change in the strainmeter. The ratio of the resistances of the two coils is nearly unity at all times. A change in ratio of 0.01 per cent normally indicates a change in length of about 4 millionths of an inch per inch. Resistance ratio changes are not affected by the simultaneous changes in the temperature of the wire since the temperature change affect both coils by an equal amount.

Temperature is measured by determining the series resistance of the two coils. This is not materially affected by changes in resistance of the two coils due to length changes because these changes are substantially equal and opposite. The series resistance in a strainmeter is about 70 ohms at 70°F and changes about 1 ohm for a change of 9°F.

The indicated length change represents the actual length change provided there is no change in temperature. If there is temperature change the indicated length change must be corrected for thermal expansion or contraction of the meter frame. In the standard meter a correction of 7.5 millionths must be added to the indicated unit length change for each 1°F

The meters are usually furnished with about 30 inches of rubber covered three conductor colour coded cable attached. The green conductor is made common to two coils, the white conductor connects to the other end of the expansion coil and the black conductor to the other end of the contraction coil. The range of meters is about 0.02 inch with the range in contraction from the so called neutral (or initial) position of the meter about 2/3 and the range in expansion about 1/3 this total.

The readings are taken by Carlson Test Set. The Carlson Test Set is basically a portable Wheatstone Bridge, consisting of a galvanometer, two known equal resistances, a variable resistance, and suitable binding posts for use with 2-, 3-, or 4-conductor cable. The variable resistance is adjusted by means of four decade dials, permitting resistance measurements from .01 to 109.99 ohms and resistance ratio measurements from 0.0001 to 1.0999 to be made. Direct readings of 2-wire resistance, 3-wire resistance, 3-wire resistance ratio, or 4-wire resistance and resistance ratio may be made with the same test set by proper binding post connections and galvanometer switches. Thus the Carlson Test Set is suitable for use with all the Carlson-type instruments. Schematic wiring diagram and operating instructions are shown on the interior of the test set cover.

(ii) Number, Location and Layout:

The considerations similar to those given under 3.2.6 (ii) apply. The lead length from the strainmeter to the test set, however, must be limited to about 200 ft.

(iii) Method of Installation:

Installing strainmeters is the most difficult and precise job involved in a special instrumentation program.

A successful strainmeter installation requires that each meter be aligned with reference to the structure axis of center-line as specified on the drawings, and that the concrete surrounding the meters be identical with that in the remainder of the structure. Obviously these goals cannot be achieved completely, but every effort should be made during placement operations to satisfy the conditions so far as practicable.

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(1) *Single Strainmeters*:—Single strainmeters are usually embedded near the top of a lift. In this case let the lift be topped off and the placing crew move away.

Step 1. Dig into the area for the full depth of the instrument, and discard all aggregate over 3-inch size.

Step 2. Backfill sufficiently to provide a bed for the instrument.

Step 3. Make a hole for vertical or diagonal meters, using an electric laboratory vibrator or by driving a pointed 1½-inch diameter steel pipe into the backfilled concrete, and insert meter in the hole. Horizontal meters are merely laid flat on the prepared bed.

Step 4. Check angles, direction and depth, using the machinists protractor and level, held against the end flange.

Step 5. Carefully vibrate or hand-puddle around the meters. If job vibrators are used, care should be taken to avoid violent or excessive vibration which might disturb the meter alignment.

Step 6. Complete backfilling by hand with 3 inch maximum aggregate concrete, and hand puddling up to grade.

(2) *Interior Groups of Meters*:—More elaborate preparations and facilities are required on interior groups of meters to assure proper installations in the limited time available before the concrete attains its initial set. It is desirable that prior to the time concrete is placed in the intended meter location the meters be brought into the block and grouped according to their arrangement in the monolith, and that tools, equipment, templates, and spare meters be placed in a conveniently accessible location. One procedure for placement of a meter group is as follows:

Step 1. Place the temporary block-out frame on top of the lift concrete when it has reached an elevation about 12 inches below final grade.

Step 2. Finish off remainder of lift outside the frame.

Step 3. Explore the bed with shovels and remove cobbles over 3 inches in size to a depth of 4 inches.

Step 4. Set template, using survey points previously marked, and place meters in approximate location, running cables out beneath block-out frame and template.

Step 5. Using an electric laboratory vibrator or a pointed 1½-inch diameter steel pipe, punch holes at the proper location in the prepared bed to receive the vertical and inclined meters. The holes should be made sufficiently deep so as to contain all but about 2 inches of the instrument, and the instrument checked for vertical or slope by placing the protractor level across the flanged end of the meter. The space between

the meter case and the side of the hole is carefully filled with stiff concrete containing little or no coarse aggregate, and thoroughly compacted. Inclination of the meter should be checked with the protractor during this operation. Horizontal meters are placed in their proper location on top of the prepared bed and aligned and levelled.

Step 6. Cover each instrument immediately with hand-placed 3-inch maximum aggregate concrete until a 3-inch thick cushion is built over and around each meter.

Step 7. After the concrete has been allowed to stiffen slightly, remove the template and blockout frame, and fill the remainder of the hole with regular fresh concrete. The concrete should not be dumped or thrown into the hole, but should be placed carefully with shovels to avoid disturbing the embedded meters or damaging them.

Step 8. After the hole is filled, it may be lightly vibrated or hand-puddled to consolidate the covering concrete, extreme care being taken to avoid walking over the area or otherwise disturbing the instruments. A board cover or barrier will serve to protect the area during the lift exposure period.

A second procedure for embedding strainmeters, particularly useful for groups of 10 or more instruments, involves the use of a special spider mounting frame which simplifies alignment of the meters. Details are as follows:

Step 1. Level off a concrete bed two feet below the ultimate elevation of the lift.

Step 2. Mark the meter group location with a light board framework and finish up the lift around this locations.

Step 3. Make final assembly of strainmeters on the spiders on a plywood or wood working platform nearby the location, and lay out and bunch cables ready for embedment.

Step 4. Explore the levelled-off bed with shovels and remove cobbles over 3 inches in size near the bedding surface. Consolidate lightly with vibrator.

Step 5. Place spider and attached meters in position, and check orientation and level.

Step 6. Carefully place concrete by hand around meters, and hand-puddle or carefully vibrate within the spider with a small laboratory vibrator. As covering of the meters begins, the shoulders of the surrounding finished lift should be lightly vibrated to avoid the possibility of sudden sloughing of the sides onto the spider assembly.

Step 7. Erect a protecting wood barrier to protect the area during the lift exposure period.

(3) *Surface Strain Meters*:—Boundary groups, or surface strainmeters, consist of several meters (from 3 to 6) placed at distances varying from 3 inches to 3

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feet from a surface of the structure with each meter arranged parallel to the concrete face and in a vertical plane. The positioning of the meters at the required distance from the face, in a vertical plane and at the proper slope can be satisfactorily attained by utilizing special pipe brackets bolted or fastened to the top of the forms. Each bracket holds a length of 1½-inch pipe, swedged shut at the bottom end, at the proper distance from and parallel to the form surface. When the concrete is placed each pipe forms a hole slightly larger than the meter diameter. A shallow hole about 8 inches deep is dug around each pipe; and after the concrete has stiffened slightly the bracket screws are removed, the pipes pulled out and the meters placed in the holes. The space between the meter case and the sides and bottom of the hole is filled with mortar and carefully tamped to insure complete contact with the instrument. The depressions around each meter are brought up to grade by carefully placing concrete with shovels and hand-tamping. A board cover or barrier will protect the area while the concrete is hardening.

4. "No Stress" Strain meters :—For the purpose of measuring independently the volumetric effects of temperature and moisture changes and chemical action within a large structure, "No-stress" strainmeters are frequently provided in conjunction with strainmeter groups. This can be accomplished by embedding an ordinary strainmeter in typical mass concrete which is isolated from deformations due to loading but is responsive to the temperature, moisture and growth changes prevailing in the mass concrete of the structure.

One arrangement, used successfully on previous installations, is to provide a cavity in the lift near the strainmeter group by embedding a 3 or 4-foot length of 15-inch dia concrete pipe so that the flange extends about 3 inches above the lift surface. A concrete pedestal is constructed around the bottom of the pipe to support it and hold it in place during placement of the concrete lift, and a precast concrete cover is provided. The strainmeter is suspended in the centre of a 12 × 24 inch cylinder specimen mould (of a type which will permit easy stripping) and the mould filled with fresh concrete taken from a typical lift batch adjacent to the strainmeter group. The 12-inch diameter mould permits the use of upto 3-inch aggregate present in the concrete in the lift. At locations where 6-inch aggregate concrete is being used, cobbles larger than 3 inches are removed from the concrete placed in the cylinder mould. The day after the cylinders are cast the moulds are removed, the cavity cleaned and water removed, the cylinder placed in the cavity, and the precast cover placed and sealed.

A second scheme, used by the Bureau of Reclamation, is to embed two strainmeters, one vertically and one horizontally, near the top of a concrete lift, and then place a 3-foot diameter 3/8 inch thick steel plate over the lift surface. The plate is held two or three inches above the concrete surface by a circumferential rim or lip, which isolates the meters from the effects

of vertical load. The length change indicated by the vertical meter includes the deformation resulting from the horizontal strains modified by Poisson's ratio, while the horizontal strainmeter aids in the evaluation of the Poisson's ratio effect.

A third scheme, developed by Sarafim, involves the use of a double-walled, double-bottom copper container which serves to protect the concrete containing the meter from strains due to load, yet provides continuity of the concrete so that other volumetric changes may take place without restraint.

(iv) *Observation, Forms of Records and Sources of Error :*

Resistance and resistance ratio readings from all Carlson-type instruments may be recorded on field reading sheets (prepared in advance) similar to Fig. II-3.1-14. The established instrument numbers and symbols are printed in the first column, the previous resistance and ratio readings entered in the second and third columns, and the remaining columns provide space for recording the current measurements. By placing duplicate sheets in a clip board, suitably off set and with a sheet of carbon paper between, the readings recorded during one series of observations are printed on the second sheet as "previous readings" for the next series of observation. The value of having the previous reading readily-available lies in the opportunity thus afforded to the observer for quickly checking and comparing the current measurements, and unusual, excessive, or erroneous readings may be immediately detected. A second or check reading may then be made.

(a) *Data Record Sheets :* Individual data record forms should be provided for each separate embedded instrument. These sheets provide a place for recording the location of the instrument, original and corrected calibration constants, lead resistances, temperature correction equations, zero resistance and ratio readings, and any other pertinent instrument characteristics. Reduction and conversion of the indicated readings to the desired units of stress, strain, temperature, joint movement, or pore pressure is made on this data sheet. Sample sheets suitable for the various types of Carlson instruments namely stressmeter, strainmeter, joint-meter and resistance thermometer are given in Figs. II-3.2-02, II-3.3-03, II-3.1-15 and II-3.5-02 respectively.

(b) *Protection of Accumulated Data :* Transfer of Data: The measurements recorded on the field reading sheets should be transferred to the individual data record forms with the least practicable delay after each set of readings are made. Transcribing of these data should be carefully done, and checked at least once in order to reduce the possibility of errors. Utilizing data record forms in this manner eliminates the necessity for exposing the only copies of accumulated readings for each instrument to the possibilities of loss or damage while making observations under job conditions. The practice of using field books for recording the readings is not recommended, since column headings and instrument numbers must be inserted by

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the observer, and considerable irreplaceable data is taken on to the job for each set of readings (frequently before the results have been transferred to data record forms).

Filing Field Reading Sheets: After the reading have been transferred to data record forms, field reading sheets should be retained in the field office until analysis of the collected data has progressed to a point where any questionable readings have been detected and checked against the original recorded values.

Filing Data Record Sheets: As each individual data sheet is completed it should be forwarded to the office where the analysis is to be accomplished. This practice will provide additional protection against loss or destruction of accumulated data, since the original field reading sheets will be retained in the field office.

(c) *Reading Schedules:* General: Initial readings must be made within three hours after embedment of the instruments and continued at frequent intervals during early ages in order to measure the rapid volume, stress, and temperature changes resulting from hydration of the cement. After maximum temperatures have been attained the observation frequency may be reduced, since volume, stress, and temperature change more slowly during the cooling off period. A further reduction in reading frequency is permissible after much of the heat of hydration has been dissipated and the structure is responding to normal climatic and load cycles.

In the interest of simplicity in establishing a reading programme it is generally advisable to refer progressive decreases (or increases) in observation frequencies to "days after embedment" for each instrument rather than stage of temperature or heat loss. The recommended duration of the various reading frequencies are intended to represent minimum requirements and to serve as a guide in establishing a reading programme. In practice, changes in observation frequencies for some instruments are deferred until such time as the change may be made simultaneously for a large number of instruments.

A normal programme consists of an initial reading made between 1 to 3 hours after embedment, readings twice daily for 3 days, once daily for the next 12 days, twice weekly for the next 4 weeks, weekly during the remainder of the construction period, and twice each month thereafter.

The observation programme for boundary meter groups would be similar to the normal programme, except that the twice daily readings should be continued for the first 15 days.

Special Stress, Strain, and Temperature Readings:

In order to determine the effect of (1) daily cyclic or other comparatively sudden climatic changes, and (2) extremes of ambient air temperature on the development of stress, strain or temperature gradients

within a structure several series of continuous readings (of upto two days duration) should be made on instruments located near exposed surfaces. Each set will consist of test set readings (or a strip chart temperature record if automatic equipment is used with thermometers) at 3 or 4-hour intervals over a 48-hour period. These observation series are usually made in pairs, during the spring and the autumn seasons to define the maximum range in daily temperature cycles and during the winter and summer seasons to coincide with the extremes of air temperatures. Similar types of observations are also made on embedded instruments adjacent to insulated surfaces to measure the effect of such insulation.

Certain precautions are required in the proper maintenance of the test set.

The current activating the galvanometer is supplied by ordinary flashlight batteries contained in the test set box, and the batteries should be replaced at intervals of not more than 6 months or whenever the galvanometer needle deflections become weak or sluggish. Holding the galvanometer switch button down continuously while balancing the bridge causes an excessive drain on the batteries as well as raising the temperature of the embedded instrument resistance coils, and should be avoided.

At least once each year or whenever there is reason to suspect malfunctioning of the test set, the test set should be partially dismantled, accumulated dust and dirt removed from the decayed dial spindles, and the decayed resistance buttons and contacts arms cleaned and polished. A light film of petroleum jelly wiped on to the contacts will aid in maintaining a satisfactory contact surface.

The main source of error is the entrance of moisture in the instrument, cable specially at the splices and at the terminal board. The terminal board must be kept dry and the terminals periodically cleaned.

(v) *Analysis of Data:*

(a) *Field reduction of data:* The term "reduction of data" refers to the arithmetical operations necessary to convert individual instrument readings into significant units of stress, strain, temperature, length, or pressure. It is desirable that these operations be performed in the field office in order that possible reading errors may be detected as soon as possible, explanations for apparently unusual results more readily and promptly secured, and preliminary results made available for use by project engineers at an early date. The practice of deferring office calculations for several months until construction work slackens, or having the data reduction work accomplished at other than the project office, should be avoided.

Reduction of data calculations required for Carlson type instruments includes one or more of the following general operations: calculations of meter temperature, application of the calibration constant to

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the measured resistance ratio, and correcting the indicated results for the effects of temperature changes in the concrete and in the meter. Step-by-step explanation of these procedures is given in 'Manual for the use of stressmeters, strainmeters and jointmeters in mass concrete' by R. W. Carlson.

(b) *Final Analysis* : Since the main purpose here is to determine stress, it is necessary to subtract that portion of the length change which is due to causes other than stress. This can be done with whatever degree of refinement the conditions suggest. In limited installations, it is often satisfactory to make the only correction for which data are provided, namely, the correction for thermal expansion of the concrete. This simple correction is made merely by subtracting from the actual length change an amount equal to the free thermal expansion or contraction of the concrete, as has been done in cols. 12 and 13 of Fig. 4. of the above mentioned reference 'Manual for the use of stressmeters, strainmeters and jointmeters in mass concrete' by R. W. Carlson.

There are numerous cases where the correction for thermal expansion of the concrete need be the only correction. The most common cases are where only approximate results are sufficient and there is known to be no appreciable alkali-aggregate expansion in the concrete, many cases of stress, and especially where this stress is mainly in one direction. Also, within many massive structures there is no appreciable volume change due to any causes except stress and temperature. Shrinkage due to loss of moisture has never been detected (to our knowledge) in the centre of a thick concrete dam. Expansion, or "growth", due to chemical action is rather common, however, and one should not assume without evidence that such growth is absent.

For the thorough investigation of stress in an important concrete structure, several refinements can be made which add to the reliability of the results. In the following paragraphs are explained all of the steps for computing the most probable elastic strains from the measured length changes.

Longer Method of Computing Elastic Strains from Length Changes :

There are several corrections and adjustments which can be made to the measured length changes to determine the most probable elastic strain, which is here defined as the length change due to stress only. In major installations where there are a number of strainmeters in each location, it is advisable to resort to the longer procedure for determining elastic strain, as shown in Fig. 6 (Carlson's booklet). Thus a computation sheet such as Fig. 6 (Carlson's booklet) would replace Columns 12 and 13 of Fig. 4 (Carlson's booklet) when the longer procedure is used.

The first step in the longer procedure of converting length changes to elastic strains is to plot to a

natural scale the actual length changes. As an illustration, the length changes from Fig. 4 have been plotted in Fig. 5 (Carlson's booklet). In general, there will be a slight scatter of points and where such is the case, a smooth curve should be drawn which most nearly represents the true length changes. This will eliminate some unavoidable, accidental variations, and yield the probable length changes at all ages. Caution is urged against altering the data in drawing the smooth curve. The practice should be to draw the curve through all of the points except those which are definitely out of line due to probable mistake.

Before proceeding further, the time intervals must be chosen which will be sufficiently close together to give useful results but not so close together as to require an excessive number of computations. Time is saved by using shorter time intervals at the early ages and longer intervals at the later ages. As a guide ages which have been found suitable are $\frac{1}{2}$ day, 2, 5, 10, 15, 20, 30 days etc.

The next step is then to pick off from the smooth curves the length change for each strain meter at each of the chosen times. These values are entered as "Unit-length Change" in space provided on a work sheet such as Fig. 6, which is a sample computation sheet for a group of nine strain meters.

Provision is made in Fig. 6 for making four different corrections to the length change. The first of these is for dilatation. The volume change of the concrete at a given location is called dilatation. It is numerically equal to the algebraic sum of the length changes in 3 dimensions at right angles. Thus, if the length change of the vertical strainmeter is added to the length changes of the two strainmeters in the horizontal plane which are all perpendicular to each other, this is one measure of dilatation. But dilatation can be measured by four different groups of 3 strainmeters (where there are 9 strainmeters), and all of these dilatations should be the same. Due to the nature of concrete, however, the observed dilatations will not be equal. The small corrections to the length changes to make the dilatations equal are called the dilatation corrections. After the corrections have been made, the dilatation computed from the length changes of any 3 strainmeter at right angles to each other should be equal to the average dilatation. A fair method of distributing the error to make the dilatation correction is given at the foot of Fig. 6 (Carlson's booklet).

After entering the dilatation correction in Fig. 6 (Carlson's booklet) the next correction is for thermal expansion or contraction of the concrete. This is an addition to the temperature correction for expansion or contraction of the strainmeter frame. It is preferred to make these corrections separately, because the temperature which controls the former is that of the strainmeter itself, while the temperature which controls the second correction is better taken as the average for the group of strainmeters. There is often a degree or

more variation in temperature among the strain meters in a single group because they are not located at a point.

The average temperature for the group of nine strainmeters is written into a space at the left of Fig. 6 (Carlson's booklet) for each of the chosen times. For example, at the first time it is 94.5°F, and at the second it is 98.7°F. Now, the particular concrete in this example has a coefficient of thermal expansion of 4.85 millionths per degree F. The temperature correction is this coefficient multiplied by the temperature difference. In the first time interval, this is 4.85 multiplied by 4.2, or 20 millionths. It is the same for all 9 strainmeters.

The final step in determining elastic strain is to make the correction for Poisson's ratio. This is necessary because strains are altered by strains in other directions. These corrections are computed according to the theory of elasticity.

A fourth correction may be necessary if there is any growth or other unaccounted volume change of the concrete. When there is growth or other volume change, the amount of such growth is merely entered in the space provided, with sign reversed. Thus a cumulative growth of 20 millionths up to the end of a given interval would be entered as minus 20.

The elastic strain for each strain meter is then the actual unit length change plus the corrections, with due regard for algebraic signs.

3.3.7 Vibrating Wire Type Gauges

ELECTROMECHANICAL OR ACOUSTIC GAUGES

(i) Principle and Construction :

The action of this type of gauge depends on the principle that the frequency of vibrating wire depends on the tension on the wire and therefore on the strain.

$$f = \frac{n}{2l} \sqrt{\frac{\delta}{s}}$$

l = Free length of the cord.

n = Harmonic level.

δ = Tension in the wire.

s = Mass per unit length of the wire.

Since the tension in the wire is proportional to the elongation, Δl , the strain.

$$\frac{\Delta l}{l} = \frac{\delta}{aE} = \left(\frac{4l^2 s}{n^2 a E} \right) f^2$$

Essentially, the strain gauge is a wire which is affixed to two end pieces that are embedded in the test structure. It also contains a magnetic pick up which plucks the wire and then transforms its mechanical vibrations into the electrical impulse. Strain changes in the specimen impose proportional changes

in gauge wire tension. The interpretation is accomplished by comparing the tone, created by the vibration of the gauge wire, with the tone of a vibrating standard wire. The tension in the standard wire may be adjusted by means of a calibration screw. Matching of the frequencies is achieved either by ear by tuning out the beats or visually by a cathode ray tube.

The main advantage of this unit as compared to the resistance type is that the frequency of the wire is independent of variation in voltages, currents and resistances. Thus any change in battery voltages, contact resistance or lead wire resistance or small leakages will render the observations futile. The square of the frequency is still proportional to change in tension of the wire.

(ii) Number, Location and Layout :

The number, location and layout depends on the type of information required. In concrete dams strain meters are installed to determine the stress distribution along certain horizontal sections of the dam. For a complete determination of the stresses at any point in a general case at least six strain gauges are required. However, as a check on the observations and for a realistic estimate of dilatation corrections a group of nine strain meters is recommended. These are arranged as follows:

Meter No. 1—Horizontal, parallel to the axis of dam.

Meter No. 2—Horizontal but perpendicular to axis.

Meter No. 3—Vertical.

Meter No. 4—In vertical plane perpendicular to axis, 135° to vertical clockwise looking towards the right abutment.

Meter No. 5—In vertical plane perpendicular to the axis, perpendicular to No. 4 (45° to the vertical clockwise).

Meter No. 6—In vertical plane parallel to axis, 45° to vertical in clockwise direction when looking downstream.

Meter No. 7—In vertical plane parallel to axis and perpendicular to No. 6 (135° to vertical in clockwise direction).

Meter No. 8—In horizontal plane, 45° clockwise from upstream direction.

Meter No. 9—In horizontal plane perpendicular to No. 8 (135° clockwise from upstream direction).

In addition to the above are two 'no stress' meters required at each location. These are strain meters which are exposed to the same condition as the surrounding concrete except as to stress. These are so

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arranged inside the concrete mass that they remain free from stress and measure the deformation from all causes other than stress. Thus in the general case a group of two meters is required. For determining the stress distribution in a horizontal section at least five such locations are desirable.

If the stress distribution could be considered almost plane as in the case of dam constructed into separate short ungrouted blocks, this number could be reduced to seven at each location. Near the surface where strains due to temperature are important three gauges may be sufficient.

The layout of the gauges depends on the information sought. The following information is generally important.

- (a) Distribution of stresses in horizontal section.
- (b) Stresses near the surfaces which are affected more by temperature variation.
- (c) Stresses in special foundation features.

For getting a general idea of the distribution five rosette installations are required at each elevation. To avoid the effects of irregularities in the foundation and for ease in interpretation these should be placed not nearer than five feet from the rock interface.

For measurement of temperature effects, the strainmeter must be distributed right through the vertical section a smaller rosette of three each could be used.

For special feature individual assessment have to be made depending on the nature of the problem.

(iii) Method of Installation:

The strainmeter is supplied with unlocked plugs and output rubber covered cable, with three leads, 20 cm. long.

The installation system varies with the selected type of connection. Two different ways are adopted. The first one considers the direct burying in the concrete of the rubber cable. Such three-pole cable is covered with a special type of rubber, suitable for the concrete and for the ground. The rubber thickness is such as to grant good mechanical and electrical protection to the leads. In such a case the strainmeter is delivered with a tube extension with suitable water tight device.

Such a tube is screwed to the strainmeter on the threading 19×1 M. On the threading a little amount of manganese putty is applied to grant the necessary seal.

The tube gives full protection to the joint between the cable protruding from the instrument and the lead cable. When the connection is made, the tube is filled with insulating material and the seal ring is locked.

The tube is lined with a rubber sheath, to avoid lateral stresses on the strainmeter; at the tube end, is then fixed with a screw, a square block. On the latter is screwed the device which holds the strainmeter while installed. The strainmeter chord is then adjusted on the selected value (250 graduations or else, if considerable tractions or compressions are foreseen) and the closing plugs are locked down, after the application of manganese putty on the threadings.

The assembling device consists of a conical tube, with an inner tie rod.

The conical tube is connected to the rectangular block, by screwing on the latter the inner tie rod. In this way the strainmeter is locked to the tube. The whole is introduced in the hollow left in the concrete casting for the strainmeter installation. The tube must be kept vertical, by hand or by means of a support.

Around the strainmeter the concrete shall be carefully cast, filling any empty space and a good connection established. During such operations, continuous control from the central board has to be made, to verify the correct from operation of the strainmeter and that the chord tension does not vary too much. Then the concrete shall undergo careful vibrating action on the block which contains the strainmeter and, at last, by unscrewing the inner tie rod it will be possible to loose and pull out the conical tube. The hole shall be then filled with liquid concrete.

A second system designs the protection of the connecting cable by means of a galvanised iron tube. In such a case the strainmeter is supplied with an iron protection tube. Such a tube carries on a side an hexagon, for the tube connection to the strainmeter; on the other side it carries a flange, for the connection with the protection tube. Near to the strainmeter the tube is covered with a rubber sheath, to avoid lateral stresses.

On the threaded connection to the strainmeter a seal cement should be applied. In the inner part of the tube the line cable joint shall be housed, then the tube shall be filled with insulating material. The counter-flange shall be welded to the protecting tube.

To set up the strainmeter with this system, it will be necessary to hold and fix the tube in such a way that the strainmeter keeps its right position and direction.

The concrete casting will follow, controlling and checking meanwhile the operation and steadiness of the adjustment. Then the concrete shall be properly vibrated.

Such a system affords a better cable protection, but it is more complicated and burdensome, especially when several strainmeters with different orientation must be installed.

(iv) *Observations, Forms of Record and Sources of Error.*

For details see acoustic stress meter 3.2.3(iv).

(v) *Analysis of Data.*

Similar to that for Carlson strain meters 3.3.5.

3.4. INSTRUMENTS FOR MEASUREMENT OF PORE PRESSURE

Instruments for measurement of pore pressures are described under the following heads:

- (a) Those used in concrete and masonry dams.
- (b) Those used in earth dams.

3.4.1 Concrete and Masonry Dams

For measurement of pore pressure at the base of the structure and in the body of the concrete and masonry dam the following types of instruments/devices are used:

- (i) Uplift pressure pipes for measuring uplift at the base of the structure.
- (ii) Electrical pore pressure gauges for recording pore pressures at the base of the structures and in the body.

3.4.1.1. Uplift Pressure Pipes

(i) *Principle and Construction:*

The device essentially consists in installing a 2½ inch diameter vertical pipe at the point at which uplift pressure is required to be observed. The pipe is then led to the nearest foundation gallery either vertically, if the gallery is located just above it, or by providing horizontal connecting pipe at the lowest level for connection to the riser below the gallery. The end of the pipe in the gallery is terminated flush with the gallery floor, and fitted with countersunk plug. A ¼-inch diameter pipe extends from the main 2½" dia pipe into the gallery gutter to which are connected necessary fittings and Bourdon type pressure gauge for uplift pressure observations.

A typical arrangement of the uplift pressure pipe installation and altitude gauge installation is shown in Fig. II-3.4-01 and Fig. II-3.4-02 respectively.

The pipe is commissioned to function as uplift pressure pipe by drilling a hole 3 feet deep into the foundation through the pipe from the gallery in case of vertical pipes subsequent to the completion of the required grouting in the area of pipe location. In case of bent pipes (i.e. pipes extending to galleries by horizontal connection pipes to riser) it is obvious that drilling through such pipes cannot be done from the gallery. So in such cases, holes have to be drilled through the vertical riser from the foundation, before bends for horizontal connections are fitted. The top of the vertical riser is then permanently capped.

To prevent choking of pipes and holes drilled prior to the completion of pressure grouting in their vicinity, during grouting, the pipes are kept filled with water under pressure slightly in excess of the grouting pressure to be applied in the particular area.

(ii) *Number, Location and Layout:*

Uplift pressure pipes should be provided in cross-sections representative of the principal types of foundation strata, under the entire structure.

Disposition of uplift pipes in any cross-section should be such as to provide adequate number of points for defining the distribution pattern of the uplift as it progressively develops, as also to detect the influence of any special foundation feature under the particular cross-section enabling verification of the uplift pressure criterion assumed in the design of the structure.

A minimum of three cross-sections namely one representing the central maximum section, one each representing the abutments, should be provided in a dam regardless of its dimensions. However, in the case of large dams a minimum of 5 or more lines of uplift pressure pipes should be provided.

Generally the uplift pressure observation points at the base of a given cross-section may be located as under:

- (1) One at the axis of the dam;
- (2) One at the centre line of the main foundation gallery i.e. downstream of the main grout curtain (cut-off).
- (3) At least three more points equally spaced downstream of the pipe in the main foundation gallery.

In case, a strata, significantly different in characteristics from the main foundation formation under the cross section, traverses across the foundation formation, a pipe may be provided near the upstream and downstream boundaries of the member with the adjacent main foundation member, to ascertain its influence on the development of uplift pressure and variations of uplift gradient. For dam which have large base widths permitting provision of greater number of uplift pipes downstream of the main foundation gallery, a general spacing of 100 feet or so may be adopted.

As stated earlier, restoration of a choked uplift pressure pipe is possible only by drilling through the pipe if pressurisation with water does not achieve the desired result. Obviously drilling through choked holes from galleries, through the bent pipes is not possible and if such pipes get choked during grouting operations there is no alternative but to abandon such pipes. Thus quite a number of bent pipes may become ineffective, resulting in loss of the desired uplift data at the location of such pipes.

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The above considerations point to the desirability of providing vertical uplift pipes, by providing transverse foundation galleries in the blocks of the dam, in which uplift pressure pipes are specified to be located. The requirements of maintaining all the uplift pressure pipes in proper operative conditions, by occasional reaming or deeper drilling of holes when warranted, make provision of transverse foundation gallery essential.

(iii) Method of Installation :

As shown in Fig. II-3.4-01, before placing the concrete the lower end of 2½-inch dia, riser pipe is anchored about 6 inches above the foundation rock, by tack welding it to an offset reinforcement bar grouted in place in the foundation. The riser pipe is then extended as concreting progresses, until it is terminated in the gallery directly above it. Uplift pipes not located directly under foundation galleries are extended by horizontal extension pipes placed on an upgrade of 0.02 toward gallery as shown in Fig. II-3.4-01, which are connected to riser pipes in the respective galleries. A 2½" × 2½" × ½" tee with a 2½ inch counter sunk plug is installed at the top of each riser pipe immediately below the gallery floor. A ½-inch pipe extends horizontally from the tee to the gallery gutter where it is capped for installation of altitude gauge assembly for pressure observations when uplift develops subsequent to the drilling of the holes through the pipes and filling of the reservoir, as mentioned under (i) above.

(iv) Method of observations, forms of record etc.

Method of Observation : When the reservoir water surface reaches an appreciable level necessitating observation of uplift at the base of the dam, the plugs (CSK) are removed, and the pipes are filled with water. In case of pipes in which the prevailing uplift level is lower than top of the pipe the water will seep into foundation and stabilise at a level indicative of the actual uplift level. Depth of water in such pipes (non-flowing pipes) may be measured by means of a bell sounder or any other suitable electrical water level indicator, capable of being lowered into 1-3/8" dia. hole. It is obvious that in case of bent pipes if the water level is lower than bend-elevations the pipe will appear dry. (It is evident that under such conditions, it is not possible to observe the prevailing uplift pressures at locations of such bent pipes. This indicates the necessity of having vertical uplift pipes by providing transverse foundation gallery above the line of uplift pipes in a block).

In case of pipes which flow when the plugs are removed, the plugs are replaced and altitude gauges calibrated in feet of water are connected through a stopcock to the ½-inch pipes that terminate in the gutter (Fig. II-3.4-02). (The first set of readings is made after the stopcocks have been opened for about two weeks, to enable stabilisation of the uplift.

In the case of nonflowing pipes stopcocks should be attached and kept in the open position until pipes start flowing, as the uplift at the station exceeds elevation of the reading when the gauge should be installed.

Record readings in the standard record sheets (Fig. II-3.4-03).

Forms of Record : Standard form of record of uplift data for any dam showing the information in the form (Fig. II-3.4-03) should be got printed in advance of the commencement of observations. All the necessary information shown in the form should be completely entered to enable correct assessment of the uplift development as affected by the reservoir level, grout curtain and the operation of drainage curtain and special foundation features.

Frequency of Observation : It is usual to make fortnightly observations of uplift. When conditions stabilise, the observations may be made at monthly intervals.

Sources of Error :

- (i) Presence of air in the piping system.
- (ii) Sticking of gauge needle.
- (iii) Leaky connections.

(v) Method of analysis of observed data

To reduce uplift pressures to a common datum for comparison, uplift pressures are expressed as a percentage of net head at the line of holes.

Net head is defined as the difference between the reservoir water surface and the tailwater surface elevation or if there is no tailwater at the line of holes, the total head is the difference between the reservoir water surface elevations and the average bottom elevations of the uplift pipes, being the average of all the bottom elevation of uplift pipes in a given line.

Computations in percentage of net head and water surface elevation in the uplift pipes should be made in accordance with the instructions shown in the form (Fig. II-3.4-03).

Results should be maintained in the form of continuous plots of uplift in terms of percentages of net head and water surface for the various uplift pipes alongwith plot of the corresponding reservoir and tailwater level.

3.4.1.2. Electrical Pore Pressure Cells

(Electrical Piezometers)

Two types of electrical piezometers are in use:

- (1) Carlson electric resistance type pore pressure cell,
- (2) Vibrating wire type (Maihak or Telemac make).

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(1) *Carlson Pore Pressure Cell*: The device is essentially similar to Carlson stress meter (refer para 3.2) but the solid steel diaphragm in this case is deflected by pressure of pore fluid which enters the space under the diaphragm, through the inlet fitted with porous stone plug. The measuring unit is similar to that of the stressmeter and is actuated by the deflection of the diaphragm caused by pore pressure. The space between the porous stone and the diaphragm is filled with petroleum jelly or water before use so that the response is almost instantaneous.

When installing the pore pressure cell care should be taken to tamp the concrete well around the conductor cable to prevent the conductor from providing a leakage path and thus altering the pressure being measured.

All the precautions required to be observed in installation of Carlson strainmeters should be observed while installing the pore pressure cell.

Readings are taken by Carlson Test Set and recorded on standard data forms. Pore pressure cells of ranges upto 200 psi are available with the manufacturer.

(2) *Vibrating Wire Type Piezometer*: Refer to para on "Electrical Piezometers" under para 3.4.2.9. and catalogue of the manufacturers of the respective makes of pore pressure cells, for other pertinent information regarding installation procedure, observation technique and equipment, analysis of observed data and precautions during installation, observations, and maintenance of terminal connections and of the indicating equipment.

3.4.2 Earth Dams

3.4.2.1. Types of Piezometers

Piezometers are of the following types:

(i) *Stand-pipe*: This is a pipe driven to the ground with an open end. The bottom portion of the pipe may be perforated or a special well point may be used. Perforated well portion or well point is provided with sand filling to form a filter and thus prevent clogging of the piezometers. Water level in the pipe directly registers pore pressures. An improvement of the standpipe was made by Casagrande which consists of a small diameter plastic pipe 3/8" diameter connected to a porous cylinder installed at the bottom of the hole. The cylinder is surrounded by sand which is capped by a plug for isolation in rest of the casing pipes. The measuring plastic pipe is protected by the casing pipe. Water level measurements are carried out by an electric probe.

(ii) *Hydraulic tip piezometers*: This consists of porous stone tip fitted into a plastic cover connected by a small diameter plastic tube leading to a point outside the body of the dam where the pressure gauges are installed for measurement.

(iii) *Electrical piezometers*: All electrical piezometers involve use of a diaphragm and a strain gauge for measurement of deflection of diaphragm. The strain gauges could be of the resistance type (Carlson) or the vibrating wire type (Maihak or Telemac). The assembly consisting of the wire strain gauge and the diaphragm is completely sealed and electric leads are taken out from it. A porous stone is fitted to the diaphragm end.

An older type of electrical gauge consists of the 'Goldbeck' cell which employs air pressure to balance water pressure in order to break electrical contact made by the diaphragm. The air pressure is applied through a tube or a pair of tubes which are taken out of the dams and connected to suitable pressure gauges.

3.4.2.2. Relative suitability of different Piezometers'

The standpipe is obviously the simplest piezometer and the most rugged. Its main drawback is the large time lag hence the open pore pipe is suitable only for relatively permeable soil, ($K 10^{-4}$ cm per second), if time lag is to be kept within reasonable limits say 2 to 4 hours.

The Casagrande type piezometers is an improvement over the simple standpipe but this also suffers from relatively longer time lag as compared to the hydraulic type piezometers, the time lag being of the order of 50 hours for $K 10^{-7}$ cm. per sec. it would be of the order of 80 days for $K 10^{-7}$ cm. per sec. Special care is, however, necessary in the installation of clay plug. The open end pipe and the Casagrande piezometers can be installed even in existing dams or deep foundations as no negative pressure is involved in the pipe system. The hydraulic piezometers is comparatively more complicated than the stand pipes. Still number of successful installations have been made as reasonable care was taken in installation and the material was properly selected. The main shortcomings of hydraulic piezometers are:

(i) Dearing is frequently required as fresh bubbles always tend to enter the tubing and tips. The deairing operation is time consuming as sufficient time has to be allotted for pressure equalization after deairing.

(ii) Freezing of the pipe makes the gauges inoperative.

(iii) Temperature variations cause inaccuracy in the pressure indications especially when different parts of the tubing are at different temperatures.

(iv) Installations have to be made in advance in order to avoid connecting of tubing to level considerably higher than the piezometric elevation as such situations are liable to result in cavitation in the tubing. Tubings have to be laid horizontally because of the above reasons hence they cause obstruction to work and create continuous horizontal passage across

the dam. Due to the possibility of cavitation if tubing has to be taken out from deep bore holes, hydraulic piezometers cannot be installed after construction.

Electrical piezometers are complicated and expensive hence they have not been much used in United States. Errors arise due to leakage of current or variations in resistance when cables are spliced. Such splicing may be unavoidable in the field when cables are cut accidentally. Hence resistance type of electrical piezometers have not proved to be reliable in the long run. On the other hand, the vibrating wire type of piezometers is reported to have worked satisfactorily in Europe over the last seven or eight years as readings are not dependent on the resistance of the cable and factors such as corrosion, temperature variations or splicing of cable do not vitiate results. Under Indian conditions the strain gauge type of electrical piezometers suffer from the major drawback of the need of importation of the special equipment. Electrical piezometers have the merit of having a small time lag but this advantage may be partly lost if bubbles accumulate in the tip below the diaphragm.

The Goldbeck cell type electrical piezometers were not found to work satisfactorily in the United States because they failed either by the rupture of the diaphragm or by internal corrosion as a result of water condensation or due to the electrical fault caused by water condensation. Accumulation of the water in the tubing would also cause erratic results as the pressure gauges at the top would not read correctly the pressure against the diaphragm. It, however, seems to be possible to overcome some of these defects by using a twin tube system for flushing condensed water and by using an enclosed type of diaphragm which may not be influenced by corrosion or condensation.

3.4.2.3. Development of the Hydraulic Piezometers

The USBR developed the hydraulic piezometers in the period of 1930—40. By 1948, 10 years experience was obtained and this was summarised by Walkér & Daehn,* (See paper presented to the Second International Soil Mechanics Conference, 1948). The essential features of design were evolved by this time and further developments have already been in the perfection of details.

Basically the USBR design consists of tip from which two tubes are taken out to a gauge well where the pressure gauges and flushing and clearing system is installed (Typical layout of this type of piezometer is shown in Fig. 2.)⁷. The twin tip tube system ensures complete flushing of air. Leaks or clogging can be easily detected and sediment can also be washed away. Some attempts have been made in the past to use single tube system e.g. the installation reported by Ward, Gibson & Penman, Geotechnique, 1955. But it is found that erratic results were obtained in the single tube system due to collection of gas. This causes variation in the piezometric constant for the difference between elevation of the tip and the gauge elevation.

*Superscripts refer to number in the Bibliography.

Consequently Ward found that piezometric levels increased although pore pressures had dissipated. Single tube system used for electrical piezometers by Speedie (1948)⁸ also showed similar erratic results. It can, therefore, be concluded that twin tube system is essential for satisfactory performance of piezometers except where the measurements are made electrically except the electrical diaphragm (Goldbeck) cell piezometer which also requires a twin tube.

Improvements in the USBR type of installation were initiated by the Building Research Station, England, and the details of this system have been reported by Penman Geotechnique 1956.⁴ In the piezometers developed by the Building Research Station, deairing apparatus consists of a pressure tank in which there is a balloon which can be pumped up with air to maintain a steady circulating pressure. This arrangement also prevents contact of the deaired water with the air. The apparatus was developed in its final form by about 1951 and it has been successfully used for a number of installations in England and abroad. The Building Research Station apparatus also permits direct observations of the air escaping from the pipelines as it bubbles through a transparent upper tank.

In the earlier USBR installation where the ordinary porous type stone was used, air could easily pass into the tubing and deairing would be more difficult. When the old type of porous stone are used an air pressure of only 1 psi is required to overcome capillary tension of the water in the porous stone so that at a greater differential pressures, air could pass from the foundation into the piezometers. Evidently the piezometers do not measure water pressure alone. Hence the specifications for the porous stone have been revised and ceramic with pore diameter of 1.5 to 2.0 microm, air entry value of 22 psi and permeability 2×10^{-7} cm/sec. is specified. This type of porous stone is now adopted by USBR as well as Building Research Station, England.

In order to minimise time lag, the expansion of the tubing and the pressure gauge for unit pressure change in this system has to be kept within permissible limits. Specifications for this purpose have been evolved (see Penman "Geotechnique" 1956).

3.4.2.4. Plastics for Piezometric Tubing

The plastic used for piezometric tubing is a critical factor governing the reliability and long term performance of piezometers embedded in earth dams or in foundations. Appendix I—3.19 of part I of this report includes a specification for the plastic material which is based on the experience of the U.S. Bureau of Reclamation. This specification requires that the tubing be manufactured from Polyvinylidene Fluoride (PVNC). This material has the advantage of low water loss, and the wall thickness of 1/16" is sufficient for withstanding crushing or accidental cutting by sharp rocks or tools during installation. Experience at U.S.

installations indicates that PVNC tubing can be expected to perform satisfactorily even 20 years after installation.

Plastics other than PVNC may also prove to be suitable for piezometric tubing provided they meet the basic requirements listed in Appendix 1-3.19, Part I, or have exhibited satisfactory performance at those installations where they have been used. Included in this category of materials which hold some promise for development as suitable alternatives are nylon, high density polythelene, oriented polypropylene and polyvinyl chloride (PVC). There is need for determining, on the basis of actual performance, whether the requirements listed in the specifications at Appendix 1-3.19 of Part I should be fully adhered to for the climatic conditions in India or other local factors. For example, the necessity of satisfying the temperature requirement of the cold bend test or the rupture strength of the tubing are matters which call for further examination.

(a) *Nylon*: Nylon is reported to lose water in form of vapour. Diffusion is dependent on the difference in vapour pressure or osmotic potential across the walls of the tube and is governed by vapour pressure outside the tube.

Imperviousness to air is, however, an important advantage of nylon tubing. Nylon is more robust than polythelene but it is expensive. Change of volume of nylon due to internal pressure is smaller than that of polythelene and this reduces response time of piezometer. Ideal material thus seems to be nylon coated with polythelene (Bishop 1964).³⁷

(b) *High density polythelene*: This is understood to be free from the defects of low density polythelene described in a subsequent paragraph. However, its acceptance as standard tubing must await field trials.

(c) *Oriented Polypropylene*: This seems to possess the desirable characteristics of PVNC material. Laboratory tests indicate that it has comparable or better strength on bursting, a slightly lower water permeability and it is workable at lower temperatures than PVNC. However, the acceptance of polypropylene as a standard tubing for piezometer installations must await its successful installation under varied conditions (Daehn 1962)³⁴.

(d) *Polyvinyl chloride*: This material has been used for some of the recent installations in India (e.g. Bor, Nalganga) and its major defect is that it is rigid and does not satisfy the cold bend test at 25°F. As service experience of this tubing is rather short (less than five years) some doubt exists regarding its long term performance. It is understood that Polyvinyl Chloride (PVC) tubing contains some plasticiser which is liable to evaporate by continuous exposure to sun (ultraviolet rays) and air. This may result in embrittlement of the tubing with age. It may be possible to prevent embrittlement by avoiding exposure to sun

prior to installation. Once the tube is installed in place some embrittlement may be permissible and generally there is no exposure to sun in the terminal wall.

Plastics which have been tried and found unsuitable for piezometric tubing include Cellulose Acetate Butyrate (CAB) and Low Density Polythelene.

(e) *Cellulose Acetate Butyrate CAB*: A major drawback of the former is the high water loss which amount to 0.0057 cubic inch per foot of tubing i.e. one pt. for 50,000 rft of tubing per day at 100 psi. Attempts at reducing water loss by filling with 50% ethylene-glycole water solutions failed due to development of osmotic pressure and floc formation which resulted in clogging of tubing (Daehn 1962).³⁴

(f) *Low Density Polythelene*: The low density polythelene has an advantage of low rate of water absorption, low water vapour permissibility and low unit price. The principal defect of this material is that it is too soft, number of failure have been reported due to cutting during installation or crushing by angular stone pieces (Daehn 1962). In India, this type of tubing was used in some of the U.P. dams viz. Nanaksagar and Jirgo (Gupta and Sharma 1964). Apparently they have performed satisfactorily but no definite information is available regarding checking of the tubes after installation with regard to leakage. According to Bishop (1964) diffusion of air can take place through walls of polythelene tubing. In view of these defects i.e., crushing and diffusion of air, low density polythelene tubing is not recommended for general field use.

3.4.2.5. Number, Location and Layout of Piezometric Installations

The following principles govern the selection of cross sections for piezometric installations:

(a) The principal types of foundations strata met with must be represented especially strata of low strength or strata liable to behave unsatisfactorily e.g. loose sand deposits vulnerable to vibration.

(b) Different types of sections adopted for the embankment in various reaches must be adequately represented.

(c) When seepage is a problem, the number of cross-sections selected will depend on the heterogeneity and complexity of the foundation profile.

When the embankment is subjected to rapid fluctuation of water levels on the upstream and downstream or two wave attack, a study of the hydraulic phenomena is necessary for selection of cross-section for piezometric installation.

Number of tips or standpipes installed in each cross-section must be adequate to provide the data required for the special features studied e.g. (i) at least three stand-pipes must be installed downstream of a grout curtain and two on the upstream so that the

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standpipe levels combined with upstream pond level and tail water level would give definite indications of the seepage gradient upstream and downstream of the curtain. (ii) When phreatic line is required to be observed, closer concentration of piezometers is necessary in the hearting zone and in the vicinity of the filters.

An allowance must also be made for failure of the tips to function and duplicate tips are necessary in critical areas.

3.4.2.6. Method of Installations

Preparation of detailed instructions regarding method of installations are considered necessary only for hydraulic piezometers. Electrical piezometers will be required in special cases only hence details instructions are not being drafted for the purpose of the Instrumentation Committee Report. The USBR Earth Manual⁴⁴ designation E-27 and E-28 adequately covers the method of installation for hydraulic and porous tube piezometers. The following material is added in amplification of the USBR Manual designation E-27 and E-28.

(a) *Requirement for proper functioning of piezometers:* In the field, partially saturated soils are often met with. In such cases, pore air pressure and pore water pressure must be measured separately and the corrected equations for effective stress in partially saturated soil must be used. In dams, however, the critical failure conditions arise when the degree of saturation exceeds 90% and it is sufficient to measure the water pressure alone. All the same the measuring device must be able to exclude air and ensure measurement of water pressure alone. Some of the earlier piezometers did not satisfy this requirement as porous stones of a suitable design were not used. The observations are liable to be erroneous in such cases even if air is removed from the measuring pipes. It may also be very difficult in such cases to exclude the air unless a fine porous stone which excludes air is used. All measurement of pore pressures involve inflow or outflow of pore fluid. Hence piezometric measurements necessarily involve time lag which is inversely proportional to the permeability of the soil. Principal problem of design of piezometers is, therefore, to minimise the time lag, to ensure exclusion of air and separation of water pressure from air pressure. A good installation must admit of easy and simple checking of clogging and leaks, presence of air pockets, variations in the calibration of pressure gauges. The instrument must be rugged enough to withstand handling in the field and exposure to atmospheric temperature and humidity variation.

(b) Check lists for installation :

Check list for hydraulic piezometers:

Testing of tubes	USBR earth manual (1963) p. 630 par. 5(b)
Testing of tips & joints for leakage	USBR earth manual (1963) p. 629 par. 4(b)

Boiling tips for fifteen minutes to remove air	USBR earth manual p. 629 par. 4(c)
Marking	USBR earth manual p. 630 par. 5(c)
Leaving slack in the tubing	USBR earth manual p. 636 par. 8(f)
Backfill to be free of stones	USBR earth manual p. 636 par. 8(e)
Avoiding reversal of grades (to prevent airlock)	USBR earth manual p. par. 8
Checking valves for leaks	USBR earth manual p. par. 18(b)
Filling gauges with fluid	USBR earth manual p. 648 par. 12
Remove air by circulating warm water before installation	USBR earth manual p. 635 par. 8(d)
Flushing with water to which aerosol copper-sulphate is added	USBR earth manual p. 648-649 par. 13

(c) *Installation procedure followed by Building Research Station, London is as under:* When the fill had reached a level about 2 ft. above the intended position of the tip, a trench 2 ft. deep was dug from the tip position to the gauge house and lined at the bottom with about 3 in. of puddled clay. The tip was laid near its correct position and temporarily sealed using two rubber O-rings clamped between steel plates. The connecting tubes were laid in the trench to the gauge house where they passed through the wall in a short length of reinforced rubber hose to protect them from the effects of relative settlement between the fill and the wall. The tubes were connected to their gauge board and the tubes and tip de-aired under a high circulating pressure upto 100 lb/in² to minimise the time required.

The fill at the bottom of the trench was then skimmed clean, the sealing clamp was removed and the tip placed with the porous disc against the fill. The level was noted and the tip was covered with fill free from stones, the polythene tubes adjacent to the tip being buried in puddled clay. A 3 in. layer of puddled clay was then placed over the tubes in the trench, and the trench was partly filled by hand with fill free from large stones, compacted with hand punners. It was then completely filled and compacted by a bulldozer or dumper.

The puddled clay contained some crushed stones, and it was thought that under high pressures these might damage the polythene tube. Samples of the puddled clay containing length of polythene tube were therefore consolidated in the oedometer under the maximum overburden which would be developed in the fill. The tubes were found to remain open during the tests, and no damage could be seen when they were subsequently removed.

3.4.2.7 Observations, Forms and Records and Source of Error

This aspect is also adequately covered by USBR earth manual (1963) designation E-27 and E-28. The following material is added in amplification of the U.S.B.R. earth manual.

SOURCES OF ERROR IN MEASUREMENT

The principal sources of error in determination of ground-water levels and pressures are summarised in Fig. II-3.4-04 and some further details are presented in the following paragraphs:

(a) *Hydrostatic time lag*: The time required for water to flow to or from the device until a desired degree of pressure equalization is attained, may be called the hydrostatic time lag. In order to reduce the time lag and increase the sensitivity of the installation to rapid pressure changes, the volume of flow required for pressure equalization should be reduced to a minimum, and the intake area should be as large as possible.

(b) *Stress adjustment time lag*: The soil structure is often disturbed and the stress conditions are changed by advancing a bore hole, driving a well point or installing and sealing a pressure measuring device, and by a flow of water to or from the device. A permanent and/or transient change in void ratio and water content of the affected soil mass will then take place, and the time required for the corresponding volume of water to flow to or from the soil may be called the stress adjustment time lag. The apparent stress adjustment time lag will be increased greatly by the presence of air or gas bubbles in the pressure measuring system or in the soil; see item 6 to 8 Fig. II-3.4-04.

(c) *Seepage along conduits*: As shown in the figure, such seepage may increase or decrease the pore-water pressure in the soil at the bottom of the hole or at the intake for a pressure measuring device. Even under regular ground-water conditions seepage may occur in closed systems with attached manometer or pressure gauge, and it will always effect experimental determination of the time lag of the system and of the permeability of the soil. To avoid seepage, the entire piezometer or the well point is often driven into the soil; but this method causes increased disturbance on the soil and in many cases it is also desirable to surround the well point with a graded sand filter. When the well point is installed in an oversized bore hole, the space between the standpipe and the well of the hole must be sealed above the well point, preferably in a fairly impermeable stratum. Puddled clay, bentonite mixture and cement grout have been used for sealing, but it is not always easy to obtain a tight seal and at the same time avoid stress changes in the surrounding soil because of swelling of the sealing material.

(d) *Interface of liquids*: To avoid corrosion or inactivation and damage by frost, monometer and pressure gauges and the upper part of piezometers may be filled with kerosene or other oils. The difference in specific gravity of water and the liquid used, as well as the position of the interface, must be taken into consideration in the pore water pressure. However, when observations are extended over long period of time, the position of the interface may change because of evaporation and/or leakage and be difficult to determine. If the interface is in the wall of a well point with very fine pores, or in fine-grained soil outside the well point additional and considerable errors may be caused by the menisci formed in the pores and by the difference in surface tension of water and the liquid in the pipe and well point.

(e) *Gas or air bubbles*: Gas or air bubbles can effect piezometric observation due to the following reasons:

(i) Changes in the average specific gravity of the water column from the top to the pressure gauge of the water surface in the standpipe. Small bubbles adhering to the walls of the pipe or tubing do not obviously cause any error in the pressure indication so long as the water column is continuous.

(ii) Discontinuity in the pressure due to air bubbles which completely fill the tube or pipe especially when there are changes in the diameter of the pipe. The pressure differences due to surface tension differs at surfaces of different radii which causes a difference in the pressure on opposite sides of the bubble, see (6) fig. II-3.4-04.

(iii) Air bubbles increase the compressibility of the fluid and hence the time lag because the volume of pore fluid drained during pressure release exceeds the initial volume due to expansion of pore fluid.

(f) *Gas bubbles in open systems (i.e. standpipes)*: All the three causes operate; however, i & ii are more significant see (6) fig. II-3.4-04. Time lag is large in open system in any case hence extra time lag due to air bubbles may not be considered. Therefore, the interior of the pipe should be smooth downward protruding edges or joints should be avoided, and the diameter of the pipe should be large enough to cause the bubbles filling the cross section to rise to the surface. These requirements are fulfilled by use of seamless and jointless plastic tubing and when the inside diameter of such tubing is 3/8m. or more.

(g) *Gas bubbles in closed systems (hydraulic piezometer)*: Air or gas bubbles in a closed pipe connected to a manometer or pressure gauge will increase the time lag, although small gas bubbles adhering to the walls of the pipe, will not effect the stabilised pressure indicated by the gauge. Air or gas bubbles

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below the gauge connection and filling the entire cross section of the pipe will influence the indicated stabilised pressure. The pipe should be provided with an air trap and outlet valve at top, and should be smooth, without protruding joints, and of a diameter large enough to permit free rise of gas bubbles. Facilities for occasional flushing should be provided and the entire installation should be composed of materials which do not cause development of gases through electrolysis.

(h) *Gas bubbles in electric piezometers*: Gas entrapped in the water-filled space below the diaphragm of an electrical pressure cell or in the perforated cover plate or porous stone will not influence the ultimate pressure indicated but will greatly increase the time lag of the pressure cell. It is conceivable that a material accumulation of gas below the diaphragm may cause the time lag of a hydrostatic electrical pressure cell to be considerably greater than that of a closed piezometer with attached manometer or Bourdon pressure gauge.

(i) *Gas bubbles in soil*: Air and other gases are often entrapped in the pores of the soil, even below the ground-water level, or dissolved in the water. When the gas bubbles migrate to and clust around the well point or are released therefrom solution in the water, the time lag will be increased on account of volume change of the gas and because the gas bubbles decrease the permeability of the soil. The well point should consist of materials which do not cause development of gases through electrolysis. It is also advisable to avoid an excessive decrease of the hydrostatic pressure inside the well point surrounding soil, since a decrease in hydrostatic pressure may cause release of gasses dissolved in the water.

(j) *Sedimentation and clogging*: Sediment in the water of the standpipe or piezometer will ultimately settle at the bottom of the pipe. An open ended pipe with a perforated end section and surrounded by sand is better. In a solid porous well point even a thin film of sediment can increase the time lag considerably. Hence the pipe should be filled with clean water. Facilities for cleaning and flushing are, therefore, essential for all piezometers. An outward flow of water from the pipe and well point may carry sediment in the pipe into the poser of the walls of the point or of the surrounding soil and may thereby cause clogging and a further increase in time lag. Therefore, and in so far as possible a strong outward flow of water from well point should be avoided.

(k) *Erosion and development*: A strong inward flow of water may carry fine particles from the soil into the pipe, thereby increasing the permeability of the soil in the vicinity of the well point and decreasing the time lag of the installation. An initial strong inward flow of water and "development" of the well point may in some cases be desirable in order to decrease the time lag, provided the well point and pipe thereafter are cleaned out and filled with clean water.

Uncontrolled erosion or development is undesirable on account of consequent unknown changes in the time lag characteristics of the installation, and because the soil grains may cause clogging of the well point, or the soil grains may be carried into the pipe, settle at the bottom, and ultimately increase the time lag. The porosity of, or openings in, the well point should be selected in accordance with the composition and character of the soil, or the well point should be surrounded with a properly grained sand or gravel filter.

HOW TO ISOLATE ERRATIC READING FROM UNUSUAL PHENOMENON *i.e.*, HOW TO ENSURE THAT OBSERVATION WHICH CANNOT BE EXPLAINED ARE NOT DUE TO INSTRUMENTAL ERRORS?

(i) When the inlet and outlet gauge readings differ by more than five feet the most probable cause is presence of air in the system. Partial choking of tubes and leakage through tubing or joint can also cause difference between the inlet and outlet gauge readings.

(ii) *Choked tubing*: If difference between outlet and inlet gauges is greater during circulation and progressively drops after closing the system, partial clogging is indicated. The tip still can be useful but the lag time would be considerably increased. When any tube gets fully choked, water cannot obviously be circulated.

(iii) *Choking of tip*: When the reading stabilises promptly to a value of almost equal to the applied pressure and different pressure readings are obtained with varying applied pressure, it can be concluded that the tip is clogged (choking of the tip can be established only after continuity of the tubing is established by pumping through tubing).

(iv) *Leaks through tubing and connections*: This gives rise to difference in inlet and outlet gauge readings and difference in the quantity of water pumped through inlet tube and the quantity discharging from the outlet tube. Obviously a clear indication is not possible of leakage when the tip intercepts highly pervious soil. In such cases the plastic tube can be converted into a porous tube piezometer.

(v) Inaccuracy in gauges is not uncommon and gauges must be checked by mercury manometer and by the master gauge frequently. Otherwise difference in outlet and inlet gauges may arise merely due to errors in gauges.

(vi) The reading should be considered erratic when the lagtime is totally inconsistent *i.e.*, 30-100 times of 1/30 to 1/100 of a calculated lagtime (or lagtime measured in laboratory test by Penman's method) based on consolidation characteristics of the material surrounding the tip (based on theory developed by Hvorsleo and Gibson).

SUGGESTIONS FOR ADDITIONAL DATA TO BE ADDED TO USBR FORMS OF RECORD

USBR forms are adequate except that they do not provide for observations on the lagtime and relevant

date regarding consolidation characteristics of material surrounding the tip. These data are essential to verify whether the tip is behaving normally. Similar checking is essential for standpipes also.

3.4.2.8 Analysis of Data

Refer USBR Earth Manual (1963) p. 652 and p. 654.

3.4.2.9 Electrical Piezometers

The reliability of hydraulic piezometers has been proved over a prolonged period by the satisfactory performance of USBR & Building Research Station (England) Installations. Many of the short-comings of hydraulic piezometers have been overcome by use of a fine ceramic porous stone of low air entry value. A major drawback of the hydraulic piezometers is the inability to transmit pressures when the tubing passes through a zone of negative pressure low enough to cause cavitation. Electrical piezometers were first developed to overcome this defect. Basically electrical piezometers fall into two types:

- (a) Diaphragm type in which the pressure is measured by make and break of an electrical contact point attached to a thin flexible diaphragm and by equalising air pressure on one side of the diaphragm with the pore pressure.
- (b) Pressure cell type of electrical piezometers consist of strain gauge attached to a stiff diaphragm. The strain gauge is either electrical resistance or vibrating wire type.

The diaphragm type of piezometers have given rise to erratic readings due to rupture of the diaphragm and due to condensation of moisture causing leakage of the current. Condensation of moisture can be prevented by pumping hot air to a twin tube system (see Miltons Speedie, p. 287-294, volume I, International Soil Mech. Conference 1948)⁴⁶.

The risk of rupture of the thin diaphragm always exists. The main advantage of the diaphragm type of electrical piezometer is its simplicity and the capacity to record pressures even when negative pressures may arise in the connecting tubing.

The principal merit of pressure cell type of piezometers is their quick response. They are, therefore, ideally suited for structures involving rapid fluctuations in pore pressures such as break-waters or dams subjected to earthquake or vibration. These instruments require care and are subjected to all the errors which are common to electrical or acoustical strain gauges.

Engineers have been inclined to believe that air may not enter the porous stone of electrical piezometers if a fine grained porous stone is used and the tip is fully saturated before installation.

Bishop (R.4, volume II, p. 47, Q.29 Eighth International Congress on Large Dams 1964)⁴⁷ has, however, proved that air (if present in partially saturated soil) will diffuse through the saturated filter after lapse of time even if a stone of relatively low air entry value is used.

The good agreement between the electrical and hydraulic piezometers for the Serec Poncon dam seems to be due to their use for a fill of low plasticity placed wet of optimum. The degree of saturation in such a fill must have been high and the difference between the pore water and pore air pressure may have been small due to the low plasticity of the core material. It is of course possible to provide additional facilities for de-airing of the filter tip of electrical piezometers at regular intervals. This would add to further complications and involve frequent de-airing operation. The instrument would then lose its advantage over the hydraulic piezometers except for special problem involving rapid changes of pressure.

3.5 INSTRUMENTS FOR TEMPERATURE MEASUREMENTS

The following types of instruments have been in use for temperature measurements in structures:

- (i) Resistance Thermometers.
- (ii) Vibrating wire type (Telemac & Maihak Make).

Under (i) are grouped the following:

1. Carlson resistance embedded type thermometer.
2. Thermo-couples.
3. Thermistor-Thermometers.

Under (ii) are the thermo-meters manufactured by Telemac (France) and Maihak (Germany).

As explained under para 2.1.1.7 measurement of temperature in concrete and stone masonry dams is essential for control of construction programme and temperature stresses and cracking tendencies and for obtaining information which will aid in interpretation of the other components of the structural action of dam.

As temperature changes in earth dams exercise no influence on their stability and structural behaviour nor are needed for interpretation of data relating to the control of safety and structural performance of these, temperature measurements are not required to be made in earth dams. In structures of concrete or reinforced concrete temperature measurements are necessary for understanding the influence of temperature change on the behaviour of these structures. However, instruments used for recording the temperatures in concrete and masonry dams, and other structures (earth dams excluded) are the same and will,

therefore, be described under the heading 'Concrete and Masonry Dams' only.

3.5.1 Instruments for Temperature Measurements in Concrete and Masonry Dams

Of the types of thermometers used for recording temperatures, Carlson resistance embedded type and thermo-couples have been extensively used and have proved their reliability for long term observations apart from the mechanical ruggedness to withstand shocks experienced during handling and embedment.

Thermistor thermometer is of recent origin and has not found wide application.

Vibrating wire types as manufactured by the Telemac (France) and Maihak (Germany) have been widely in use in Europe.

3.5.1.1 Carlson Resistance Thermometer

The instrument shown in Figure II-3.5-01 is Carlson resistance thermometer.

(i) Principle and construction

The operating principle is based on the variations of resistance as a function of temperature. The resistance thermometer is simply a non-inductively wound coil of enamelled copper wire, enclosed in a brass case. Great care has been taken to be sure that moisture is prevented from entering the case. Any moisture that might possibly work its way along the cable and into the case, or down through the splices in the cable itself are stopped by the cable joint compound with which the instrument is completely filled. The thermometer is equipped with 30 inch of 3-conductor rubber covered cable. Two conductors are connected to one end of the coil and one to the other. By this arrangement (and a special) testing set it is possible to extend the cable to any desired length without regard to its effect on the thermometer indications. The three conductor arrangement makes the thermometer self-compensating when used with a Carlson testing set. This is accomplished by having one conductor in each of the two arms of a wheatstone bridge such that they cancel one another. The resistance of the thermometer is 39.00 ohms at 0°F. This makes it very simple to convert a given resistance reading to temperature.

(ii) Number, locations and layout

Temperature change being the principal factor apart from construction procedures in causing stress in massive concrete structures, it is imperative that temperature be measured accurately at many points in the structure, in the water and in the air.

A typical scheme would be to place thermometer in 50 × 50' grid horizontally and vertically in the block representing the central deepest portion and a

block each in abutment, representing the blocks built on steeply sloping abutments, as selected for the detailed instrumentation for study of structural behaviour of dam. A closer spacing will have to be adopted for thinner dams.

As Carlson strainmeters, stressmeters and jointmeters also provide an accurate measurement of temperature at their locations in dam, there is no need to duplicate instruments at such locations. However, at locations where vibrating strainmeters and stressmeters are used thermometers should be placed at such locations for temperature records.

A few thermometers should be installed at and near the downstream face to evaluate rapid daily and weekly fluctuations in temperature. Thermometers placed at the u/s face in continuation of the main thermometer grid will serve to evaluate lake temperatures as they vary through the year.

In order to better define the steep thermal gradients near the faces of the dam, it is desirable to place thermometers at 10' d/s of the axis and 10' inside from the face of the dam, in addition to those in grid. For measurement of foundation temperature, thermometers should be placed near the base of dam and also in holes drilled into the foundation at desired locations, for study of the effects of restraint near the foundation zone.

In order to study the effect of operating penstocks and river outlets, on the temperature of surrounding concrete mass, one line of thermometers should be installed, spacing of thermometers from the outside of pipe being 6", 18", 0.3 radius and 0.5 radius of the conduits.

For study of temperature rise and temperature in the concrete lift when placed in position, thermometers at vertical spacing of 12" may be embedded at location specially selected for such study.

(iii) Method of installation

Even though resistance thermometers are furnished in a brass case, some preparation is necessary prior to embedment. This varies with the type of embedment:

1. For embedment in the middle of a lift, it has been helpful to give each thermometer a quick dip in sealing compound, and tape with one layer of friction type for additional shock resistance and additional water proofing insurance during embedment.
2. Where resistance thermometers are to be embedded at the u/s face to measure water temperatures, additional water proofing against hydrostatic pressure is furnished by encasing the thermometer in an 8" length of grout tubing which is then completely filled with sealing compound.

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For embedment at the base of a lift, it is best to tie down some wires to the concrete of the previous lift or between two cooling pipes. The thermometer can then be wired or taped securely, to hold down thermometer so that the pressure of the mass concretes will not dislodge it.

For an installation at the top of a lift, a small hole can be dug, and the thermometer inserted and covered immediately, using the foot or a vibrator to puddle the concrete around the instrument. For accurate spacing at various heights in a lift, the thermometer can then be wired or taped securely, to hold down thermometer so that the pressure of the mass concrete will not dislodge it.

(b) *Reservoir Thermometer Installation*: At the downstream or upstream face, the thermometer can be held by hand against the form while concrete is back-filled around it. It is, however, better to tie the thermometer to the form with light wires and provide anchors on to the meter case and back into the concrete so that the thermometer will not be pulled out of the weak concrete when the form is stripped.

(iv) *Observations*

Observations on thermometers are made by connecting the thermometer cable lead terminations on terminal boards located in galleries to the standard wheatstone bridge test set through standard test lead in accordance with the manufacturer's instructions for circuit connections through standard test lead, when the observations of temperature are required. When the instrument is being embedded, observations are made by directly connecting the thermometer cable lead to the test set in accordance with manufacturer's instructions for making temperature measurements.

(a) *Schedule of observations*: Thermometers may be observed at the following intervals:

- a. Immediately after embedment.
- b. Semi-weekly for a month.
- c. Weekly until cooling is completed.
- d. Fortnightly afterwards.

Of course, where thermometers are embedded alongwith strainmeters and stressmeters, they have to be observed alongwith and at the same time as stress and strainmeters.

(b) *Form of record*: Recording of thermometers should be recorded in the space provided for the purpose under column "Resistance", against the relevant thermometer number in the printed field data form, sample of which is attached as Fig. II-3.1-14.

(c) *Sources of error*:

- (i) Low voltage of test set batteries.

- (ii) Loose connection of cable terminals on terminal panels.
- (iii) Loose connections in the test set circuit.
- (iv) High voltage may cause heating of the wire and thus affect accuracy of the reading.
- (v) *Analysis of observed data*

On receipt of the Field Reading forms in the data processing section, the relevant information should be transcribed in the Thermometer Data sheet Fig. II-3.5-02 and analysis of data should be done in accordance with the following procedure.

All Carlson thermometers are adjusted to 39.00 ohms at 0°F, and the change in resistance is exactly 0.10 ohms per degree.

Thus the temperature can be computed merely by subtracting 39.00 ohms from the measured resistance and moving the decimal place over one position. For example, at 70 degrees, the resistance will be exactly 7 ohms higher than at 0°F or 46.00 ohms. Subtracting 39.00 ohms from 46.00 gives 7.00 ohms and moving the decimal one place gives the temperature of 70.0°F.

3.5.1.2 *Thermocouples*

For complete description, method of installation observations etc. refer page 58 to 63 of the Corps of Engineers Manual EM 110-2-4300.

3.5.1.3 *Thermistor Thermometer*

(i) *Principle and construction*:

Thermistor thermometer is essentially a special type of resistance thermometer consisting of a semiconductor of sintered metallic oxide shown in Fig. II-3.5-03 (taken from the article "Temperature measurement with thermistors in concrete" Transactions Symposium on observations of structures Vol. II). Thermistors are made in several countries by well known manufacturers of electronic equipment and are available in several types. The thermistor has a high negative coefficient of resistance variations with temperatures and after being suitably calibrated can give temperatures with an accuracy of 0.01°C if very sensitive measuring instruments are used and if an individual calibration is made. However, in usual conditions an accuracy of 0.5°C is obtained. The resistance does not vary in proportion to the temperature and drifts with time. This drift, however, is smaller than 0.1°C per year and can be prevented by a heat treatment. Due to high resistance of the thermistors the errors produced by variation in resistance of the lines and the contact resistances are negligible.

The operating range of these thermistors is wide enough to cover the temperature ranges met within building construction and structural engineering. Upper limit may be 300°C.

Referring to the figures referred to above, it will be seen that the dimensions of the temperature sensitive organ, the bead, are very small. The electrical resistance is measured between two very thin platinum wires which are sealed into the bead. The semi-conductor material itself is protected by a glass casing, which increases the stability of the thermistor and which is designed to facilitate handling.

For use in large concrete structures thermometer design provides for mechanical ruggedness, and long water-proof conductors, to afford adequate mechanical protection of the thermistor and leads, made of ordinary insulated copper wire for measurements of resistance values of thermistor at relatively great distances from the gauge point, and protection of the lead joints.

To meet these requirements the thermistor is enclosed in a brass tube, surrounded by a suitable sheath e.g., a welding rubber tube Fig. II-3.5-04 (a) which protects the leads. The assembly is moisture proofed in this case first by sealing up the thermistor and the leads in a thermo plastic material, and second by a layer of rubber cement between the brass tube and the welding rubber tube. The sealing is carried out as follows:

The thermistor is soldered to the PVC insulated leads. The brass tube is filled with thermoplastic material PVC paste (plastisol G 20003). The thermistor and the ends of the leads are inserted into the brass tube to a distance of about 5 mm. from its bottom. After that the assembly is subjected to thermo-hardening at 150°C for 10 minutes so as to cause polymerisation. This method ensures efficient mechanical protection of the thermistor and reliable moisture proofing of the soldered joint, since the PVC paste and the PVC insulation of the leads are completely melted together.

The leads and contacts can be made of ordinary simple standard material, the length of the leads can be very great and can also be varied within wide limits and finally the contacts can be inserted in the measuring circuits without producing any practically important effect on the accuracy of the measurements on account of the high resistance of the thermistor. It is advisable to use thermistors whose resistance at the temperatures in question ranges from 200 to 20,000 ohms.

(ii) *Number, location and layout*—Refer to para 3.5.1.1. (ii)

(iii) *Method of installation*: Refer to para 3.5.1.1. (iii)

To ensure adequate mechanical protection of the leads from stresses due to conveyances laying and embedding, the leads can be enclosed in tubes or conduits e.g., welding rubber tubes.

(iv) *Observation, forms of record*

Observation of the thermistor resistances are made by connecting the leads to the resistance bridge of the wheatstone type. The characteristics of the bridge are determined by the requisite accuracy of measurements with due regard to the sensitivity of the thermistor to the power input i.e., the heating of the thermistor caused by the current flowing in the measuring circuit. The current flowing through the thermistor should be reduced as much as possible and at the same time sensitivity of the galvanometer in the bridge should be increased to a corresponding degree.

A typical wheatstone bridge test circuit is given in Fig. II-3.5-04 (b).

Automatic recording of temperature measurements can be adopted where continuous record of temperature is required.

Forms of Record: Ref. to para 3.5.1.1. (iv).

(v) *Analysis of Observed Data*

Temperatures are computed from the observed resistances by using the calibration curve of the thermistor in use.

3.5.1.4. *Vibrating wire-type thermometers*

Instruments based on the principle of vibrating wire transducer are quite commonly used in Europe but have not found much application in USA or elsewhere. Companies in Italy (Galileo), France (Telemac) and Germany (Maihak) manufacture vibrating wire instruments.

(i) *Principle and construction*

The vibrating wire thermometers like other vibrating wire instruments viz., strainmeters clinometers joint meters etc. operate on the principle that a change in length of the vibrating wire, pretensioned to vibrate and excited to damped natural vibration by an impulse from the Receiver, caused by change in temperature, results in proportional change in the natural frequency of the vibrating wire, measuring wire, also called the transducer element. The transducer or transmitter has two components, a pretensioned steel wire and an electromagnet. When the wire is plucked, by an impulse from the Receiver, an alternating current signal is indicated in the magnet circuit. This signal has the same frequency as the vibrating wire. An increase in tension of the wire produced by lengthening of the transducer element causes corresponding increase in frequency of the signal. Many vibrating wire meters are equipped with two electromagnets, one for plucking or sustaining the vibration of the wire, and the other for generating the signal. Maihak type vibrating wire type thermometers having measuring range of -20°C to 70°C (-5°F to 157°F) (and even higher) and sensitivity of 0.1° to 0.2°C depending upon measuring range, water tight upto 30 atmos

and 30 mm in diameter and 170 mm long are used for temperature measurements in view. Telemac thermometers of this measuring range are also used for temperature measurement. The meters are equipped with two conductor copper shielded cable, embedded in concrete and led into the reading stations located in galleries in the dam or the instrument leads are terminated in a central reading station located in a surface building constructed near the dam. Test sets used with the vibrating wire thermometers (as for other instruments of this type) for obtaining indications of temperatures are of two types, one provides audio indications and the other visual description.

The audio-type set which is usually used for field work, consists of an electro-mechanical comparator, audio amplifier and a speaker or a head phone. The comparator (Telemac) or Receiver (Maihak) is a Standard vibrating wire, similar in construction to the transducer element. The tension and frequency of the comparator wire can be increased or decreased by means of a micro-meter attached to one end of the wire.

Signals from a transducer (Telemac) transmitter (Maihak) and the comparator are amplified and fed into the speaker. Here they produce simple audible tones. To adjust the frequency of the comparator to that of the transducer, the two tones, at first, are compared individually. When the comparator has been adjusted so that its tone is approximately the same as the tone produced by the transducer signal, then both signals are compared simultaneously. When this done, beating can be detected. To obtain the desired matching of the two tones, the comparator micrometer is further adjusted until the period of the beating is about 3 to 5 seconds. Included also in this type of vibrating wire indicator is a standard tuning fork. This is used to check the drift of the comparator when readings are made over extended periods.

The visual-type indicator used with vibrating wire meters has also a comparator as one of its components. Depending upon the manufacturer, the comparator may be similar to that described above, or it may be a stable electronic oscillator. In either case signals from the comparator and a vibrating wire transducer are amplified and applied simultaneously to the X and Y plates respectively, of a cathode ray tube. These signals produce Lissajous figures on the screen. When the frequency of the comparator signal is the same as that of the transducer signal, the resulting figure will appear as an ellipse. Cathode ray type indicators are unsuitable for field use where external power is not available.

(ii) *Number, location and layout*

Refer to para 3.5.1.1. (ii)

(iii) *Method of installation*

Refer to para 3.5.1.1. (iii)

(iv) *Observation, Forms of Record, Sources of Error*

Observations of temperature are made by connecting the meters with test sets in accordance with the manufacturers instructions and recorded in the printed forms as mentioned under 3.5.1.1. (iv). The accuracy of the readings does not depend upon maintaining high resistance to ground.

(v) *Analysis of observed data*

The observed data should be transcribed into the printed thermometer data sheets and analysis of the data done by using the calibration curves or the calibration constants of the respective meters in accordance with the manufacturer's instructions.

For details refer to Telemac and Maihak catalogues respectively.

3.6 INSTRUMENTS FOR MEASUREMENTS OF EXTERNAL LOADING

The measurements involved in assessment of external loads are described in paras 2.1.2 and 2.1.2.1. to 2.1.2.5. Measurement of dynamic load due to waves earthquake and wind is excluded and measurements of hydrostatic pressure are also excluded as they do not require any special equipment. Thus the only instruments for measurement of external load that need to be dealt with in this report are :

- (i) Earth pressure cells.
- (ii) Instruments for measuring rock loads.

Techniques for measurements of rock loads are still in the stage of development and there is no experience so far in India of this type of measurement. Hence these are also excluded.

3.6.1 Earth Pressure Cells

These are further divided into two categories.

- (a) Contact pressure cells attached to structures subjected to earth pressures.
- (b) Total pressure cells buried in the earthmass.

3.6.1.1 Contact Pressure Cells

Contact pressure cells are essentially the same as stressmeters in concrete. They are subject to same limitation as stressmeters. However, due to greater deformability of earth, relatively larger deformations can be permitted than for stressmeters in concrete. Two types of contact pressure cells have been used.*

- (a) *The Goldbeck cell* : This is based on the principle of balancing earth pressure by air pressure on the opposite side of a thin flexible diaphragm.

*General report Serafim, Borges Roche (Rilem 1955 Symposium on observations of structures).

INSTRUMENTS

The accuracy of Goldbeck cell is dependent on the sensitivity of the balancing system and the rapidity with which imbalance can be corrected. In the absence of automatic control temporary deformations of the diaphragm cannot be prevented and it may be necessary to bring back the diaphragm to the original condition by application of pressure. This involves a cyclic change in the position of the diaphragm and significant error in measurement can be caused unless the cell can register with very small movement of the diaphragm. As generally make and break principle is used, a minimum gap is necessary to avoid leakage of current and sparking. Unless diaphragm is thin, it would not correctly register the earth pressure as its own resistance to deformation would have to be accounted for. Hence it is customary to provide this diaphragm which is vulnerable to damage by angular particles in the soil. Thin diaphragm may be ruptured when excessive balancing pressure is inadvertently used. Erratic readings can also be caused due to moisture condensation within the cell. The use of Goldbeck cell has, therefore, been abandoned for earth pressure measurements. There is no report on installation having been made after 1938.

(b) *Contact pressure cells*: The twin plate type contact pressure cell has been developed by manufacturers of vibrating wire instruments. In these instruments the earth pressure is taken by a stiff plate to transfer the earth reaction to a beam which deflects under the load and is in turn supported by another plate. The twin plate unit bears on the surface of the rigid structure (e.g. retaining wall). The deflection of the steel beam is measured by vibrating wire instruments. Because of the sensitivity of the vibrating instruments, deformation of the measuring plate is very small.

According to studies carried out at U.S. Waterways Experiment Station (Technical Memorandum 201-I soil pressure cell investigation), the deformation of cell measuring earth pressure in sand should not be more than $1/2000$ of the diameter. For 10" dia. cell (300 mm.), this requirement can be satisfied if the deformation is within $1/200$ th of an inch. The vibrating wire instruments of the twin disc type can, therefore, be considered to be inherently capable of recording the earth pressure with reasonable accuracy. However, these instruments are subject to the same limitations with regard to the accuracy, difficulties of manufacture and installation as the vibrating wire strain gauges.

(c) *The Carlson type stressmeters* can also be used for measurement of contact pressure. The only difference being that larger movements can be permitted without loss of accuracy as compared to the stress meters for concrete. It does not seem to have any special advantage over the twin plate vibrating wire type of instruments. On the other hand it has the disadvantage that the measuring errors increase and sensitivity decreases when cable leads are long. Hence the use of Carlson type of stressmeter for earth pressure measurements would have to be restricted to installation where the length of lead is within 500 ft.

3.6.1.2 Measurements of stress within the Earthmass

Computation of stresses from strains is difficult in soils because of the complex mechanical properties of soils, in reversibility of the stress phenomena and the influence of creep and consolidation. When lateral yield is absent, pore pressures are measured reliably in the absence of stress reversal, strain computed from cross arm measurements can give fairly good indications of stresses. As these conditions are realised only in relatively small part of the earth mass, direct measurement of stresses becomes necessary whenever it is desired to investigate the condition of stress within the earthmass.

A satisfactory total earth pressure cell must be thin and rigid as to cause minimum disturbance in the stress pattern.

Recent developments in this direction seem to be promising but the reliability of these instruments have not yet been sufficiently confirmed.

At the Serreponcon Dam the new conception of thin and rigid measuring capsules was used which has practically constant deformation all over the surface. This deformation is very small so that significant redistribution of stresses would not occur within the earthmass. Details of these instruments have not been published but the following general information is available.

Inside the pressure cell the normal stress is transformed into tension over a transverse elastic membrane the deformation of which are measured by a vibrating wire. The apparatus is made of special corrosion resistant brass.

The cell is mostly hollow so that the pore fluid penetrates all over; hence the pore pressure would have very secondary influence.

From review of data supplied by the manufacturers and R.3 (ICOLD Q. 29 Vol. II)/1964 Barge, Post and Huynh, it is seen that out of seven instruments three seem to have behaved satisfactorily at the Surreponcon Dam in France. The error may be 15-20% (by comparison with extensometer reading in the concrete drainage duct). Readings of two instruments seem to be influenced by calibration error. Two instruments seem to be erratic and hence unreliable.

At the BRS England a stable cell incorporating a vibrating wire gauge parallel to a steel diaphragm has been designed. The instruments were installed in groups of five at Balderhed and Mangla so that principal total stresses can be deduced (total number of cells at Mangla 40, Furid 10). Analysis of the data for Mangla and Balderhed is not yet available. Hence it can be concluded that the recent developments in total pressure cells are promising but the reliability of the instruments is not sufficiently confirmed.

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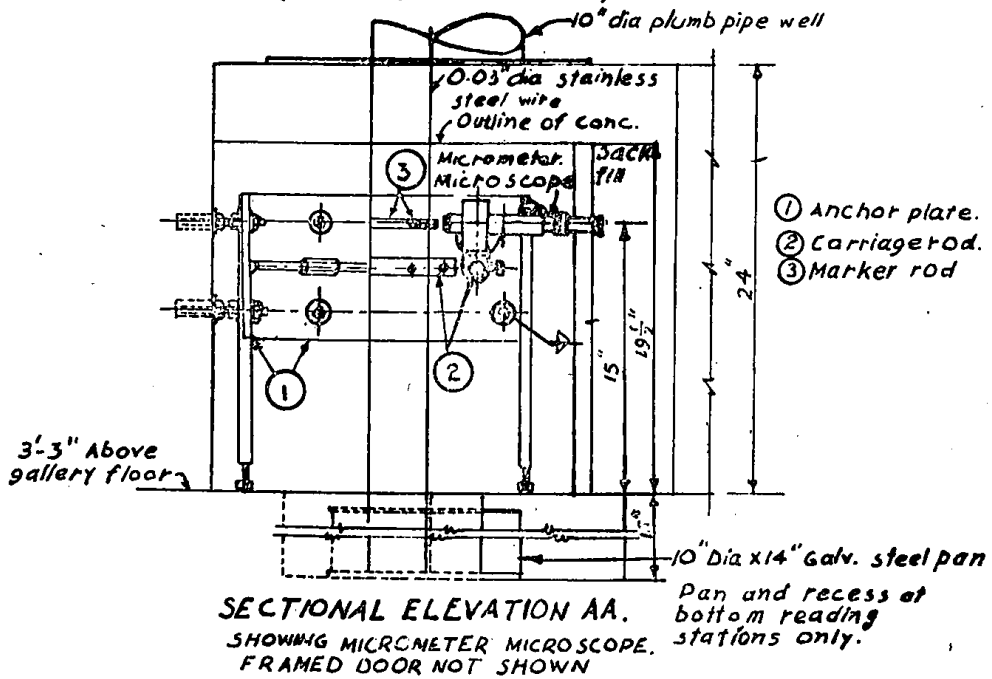
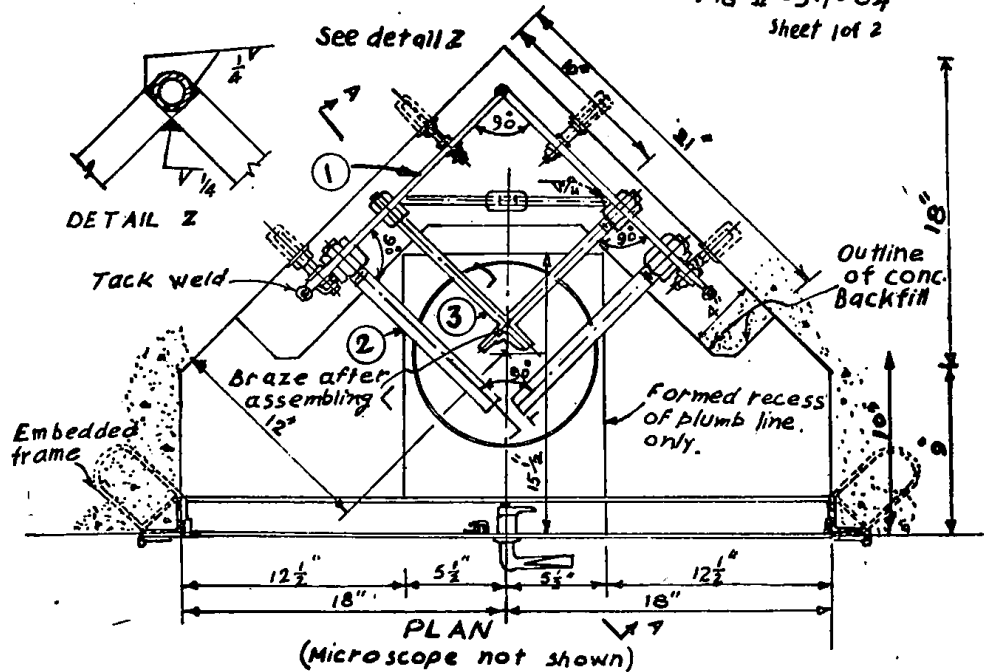
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HORIZONTAL DIRECTIONS

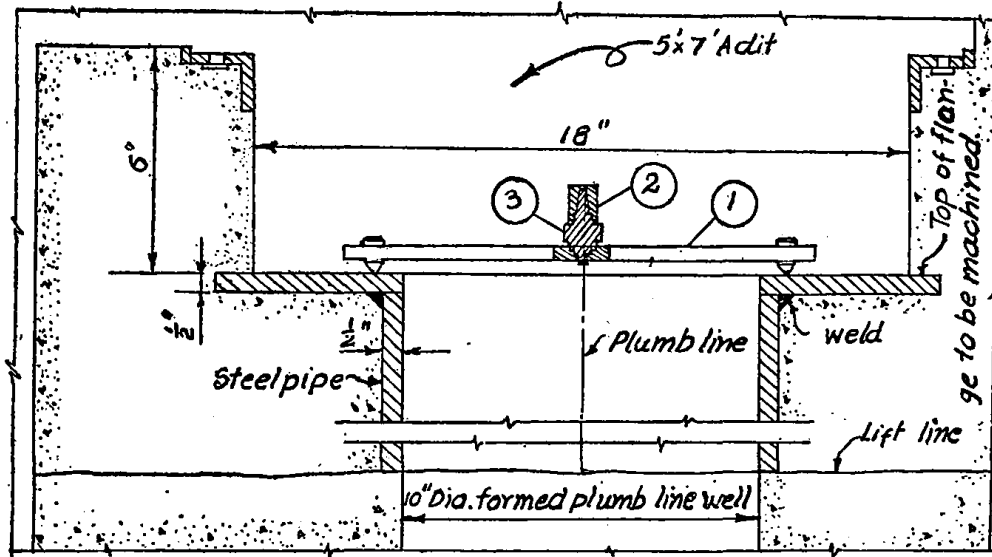
PROJECT _____ DAM _____ SHEET _____ OF _____
 STATION _____ OBSERVER _____
 H. I. _____ RECORDER _____ DATE _____
 WEATHER COND. _____ INSTRUMENT _____

POSITION TIME	STATION OBSERVED	D	R°'	1 DRUM 2		MEAN D & R ⁹'	DIREC- TION'	REMARKS
					''			
		D								
		R								
		D								
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FIG II -3-1-04
Sheet 1 of 2



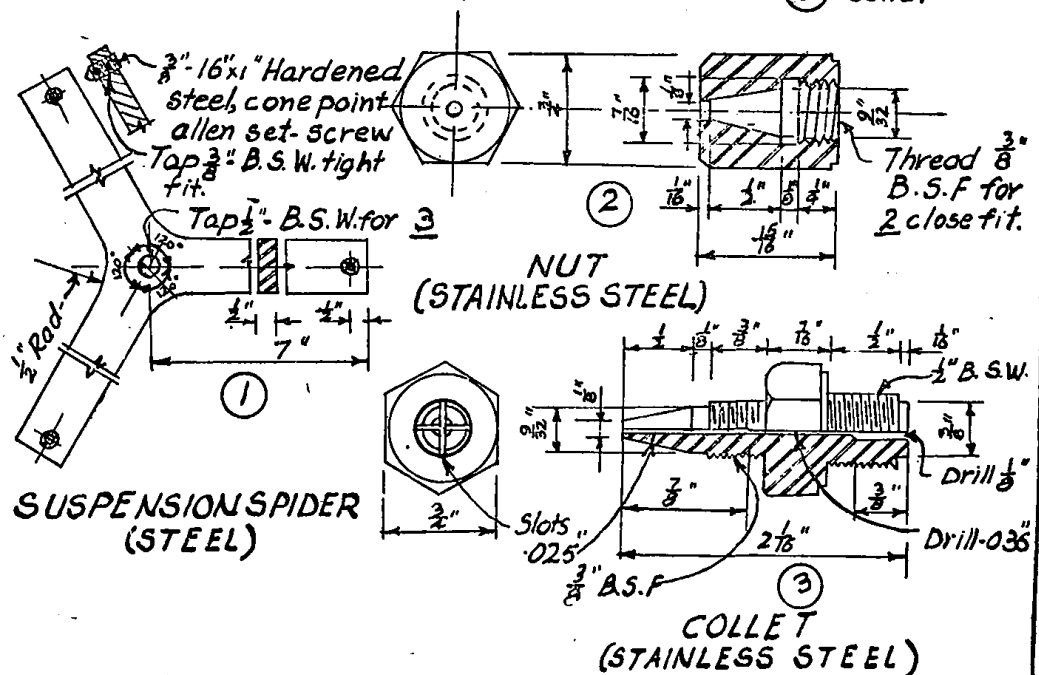
DEFLECTION MEASURING EQUIPMENT



SUSPENSION ASSEMBLY
(Cover plate removed)

LEGEND

- ① - Suspension spider
- ② - Nut
- ③ - Collet



DEFLECTION MEASURING EQUIPMENT

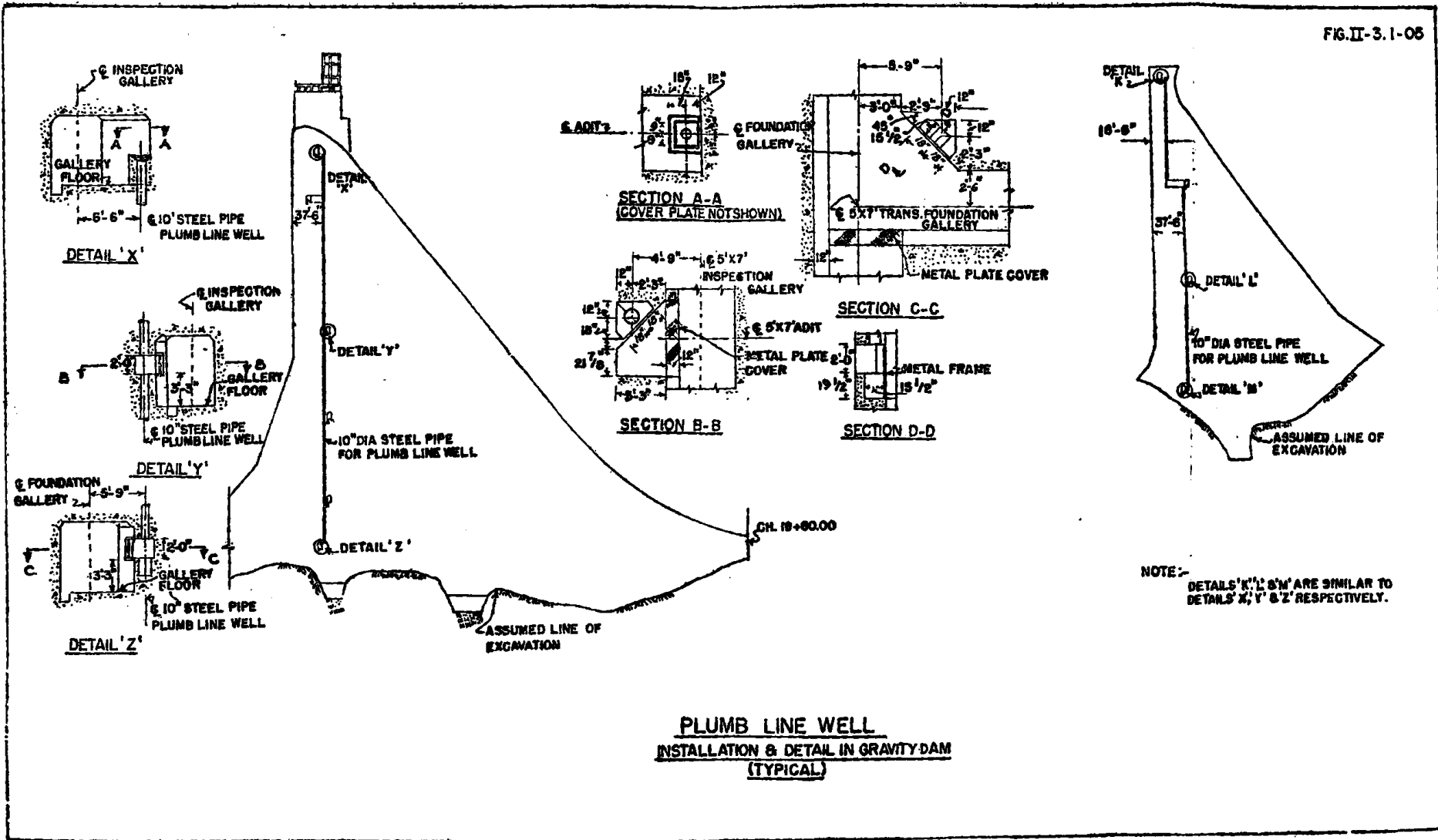
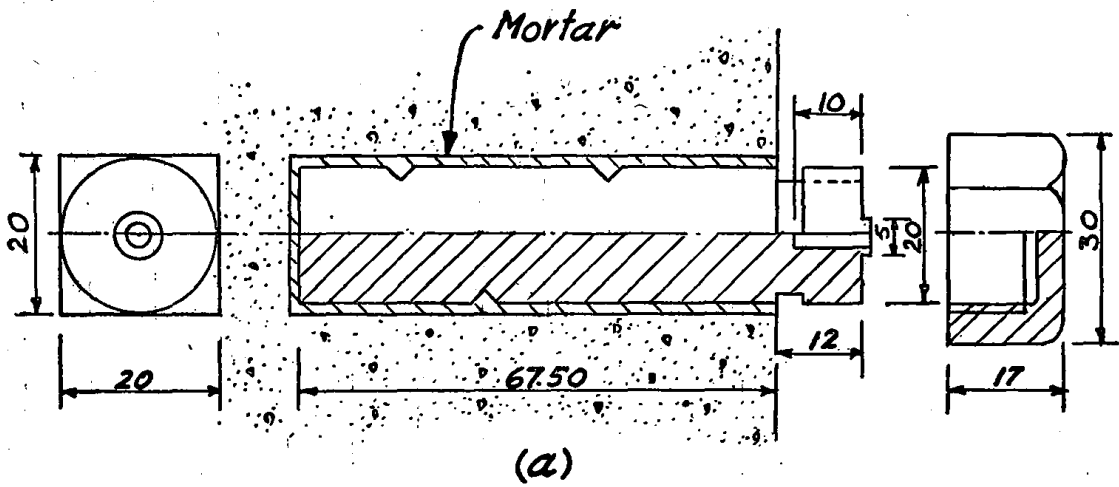
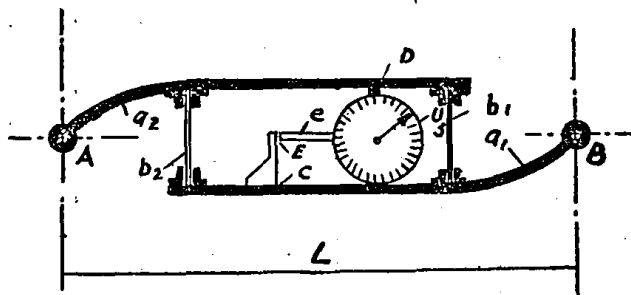


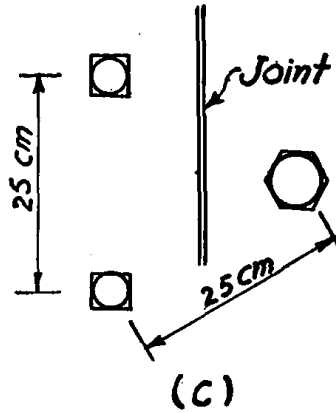
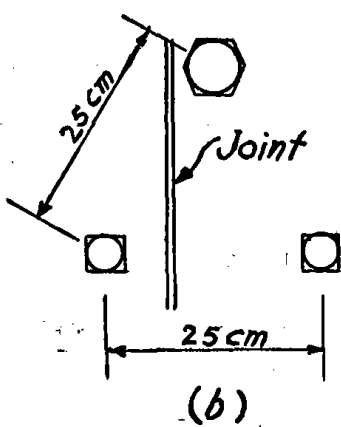
Fig. II-5-1-07



Whittemore and Marion gauge inserts

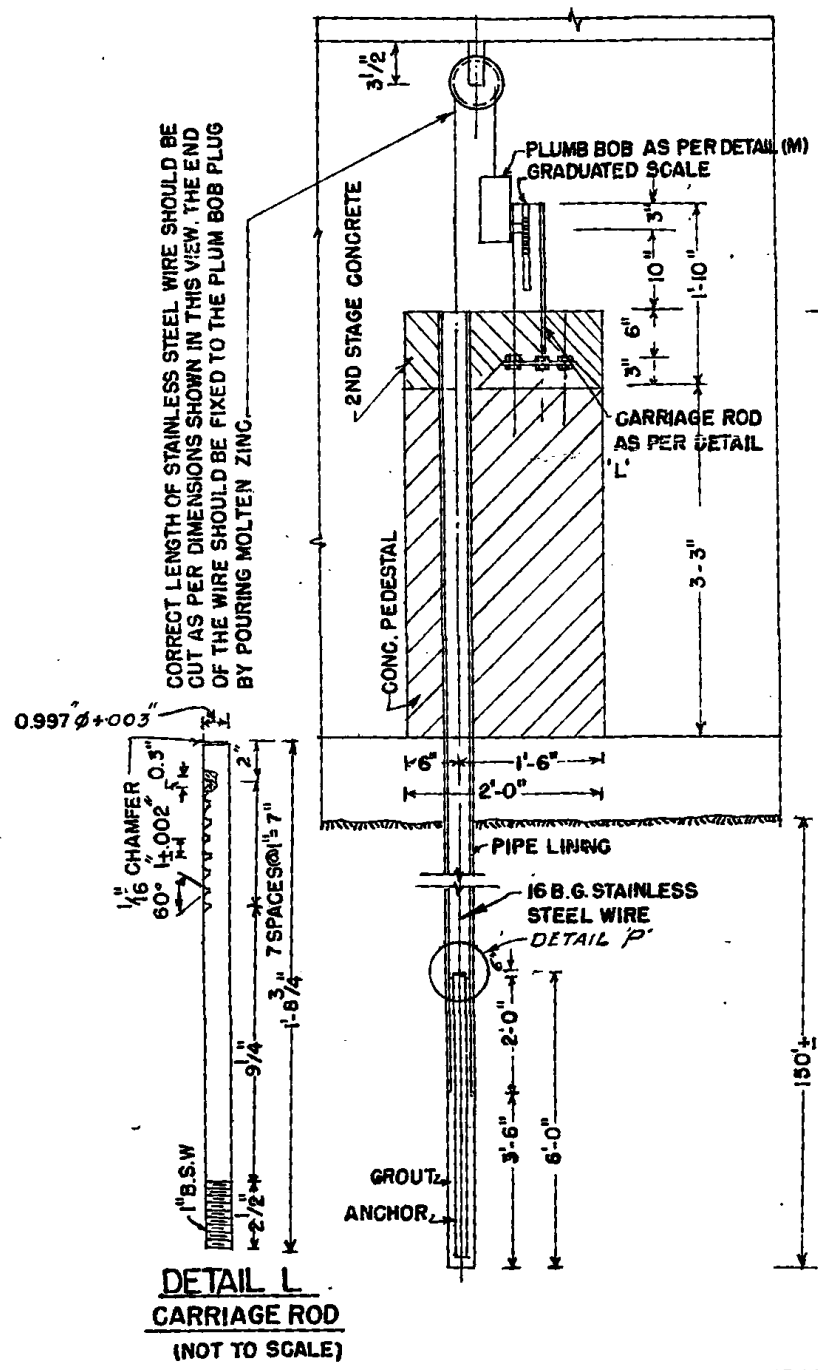


Schematic View of Whittemore type gauge

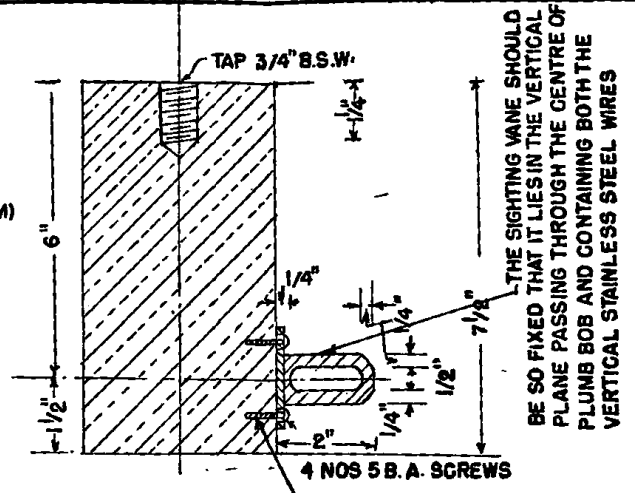


Position of the gauge inserts relative to the joint

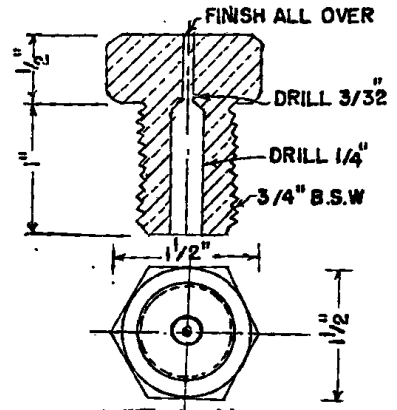
SETTLEMENT MEASURING DEVICE



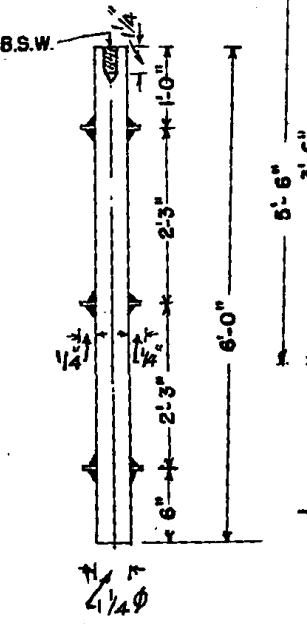
DETAIL L
CARRIAGE ROD
(NOT TO SCALE)



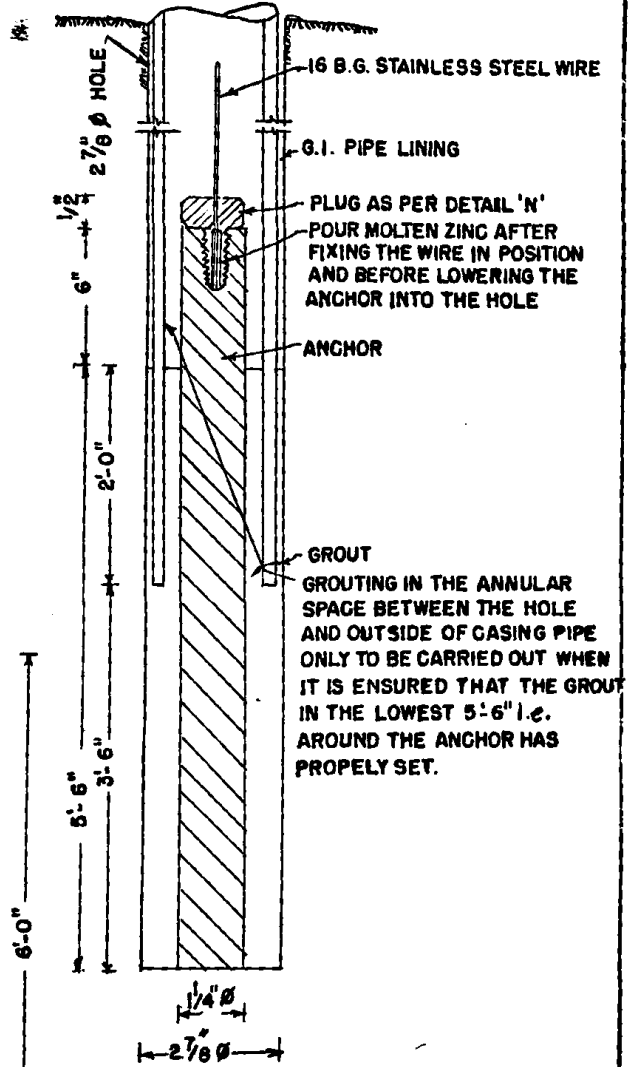
DETAIL M
PLUMB BOB



DETAIL N
PLUMB BOB PLUG

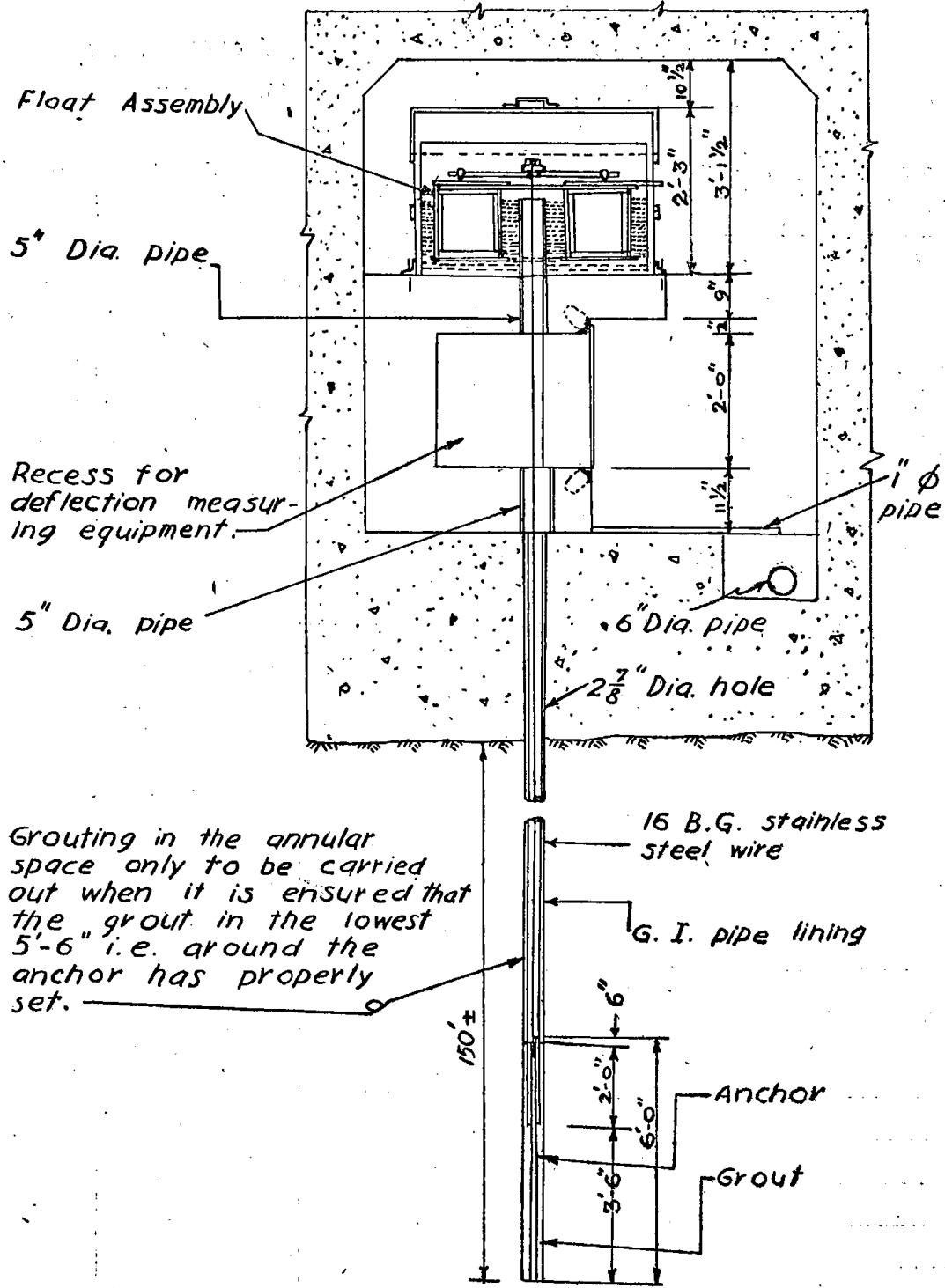


DETAIL OF ANCHOR
(NOT TO SCALE)



DETAIL P

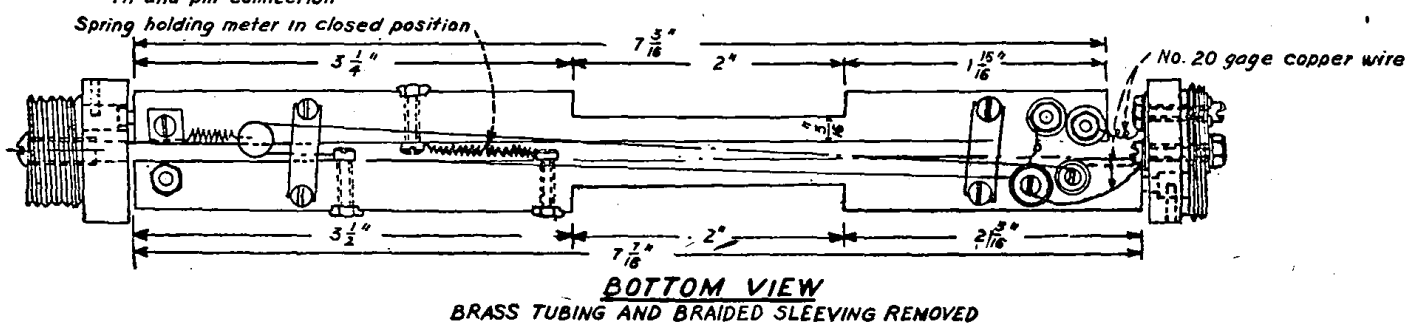
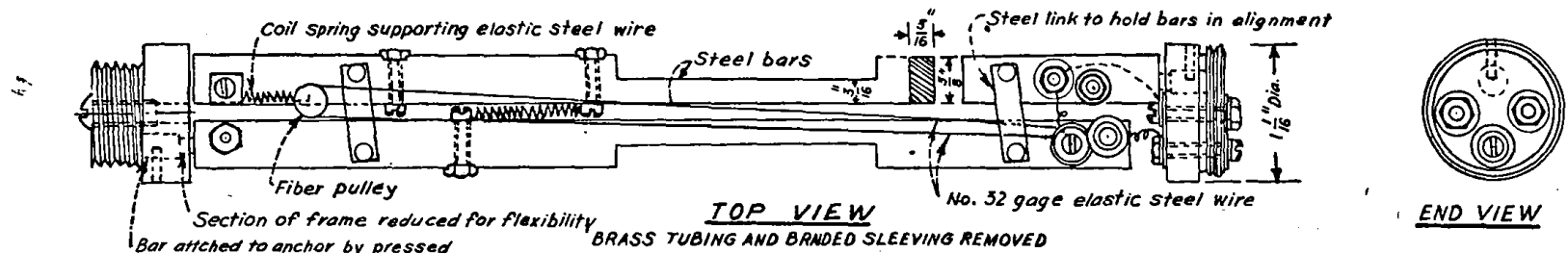
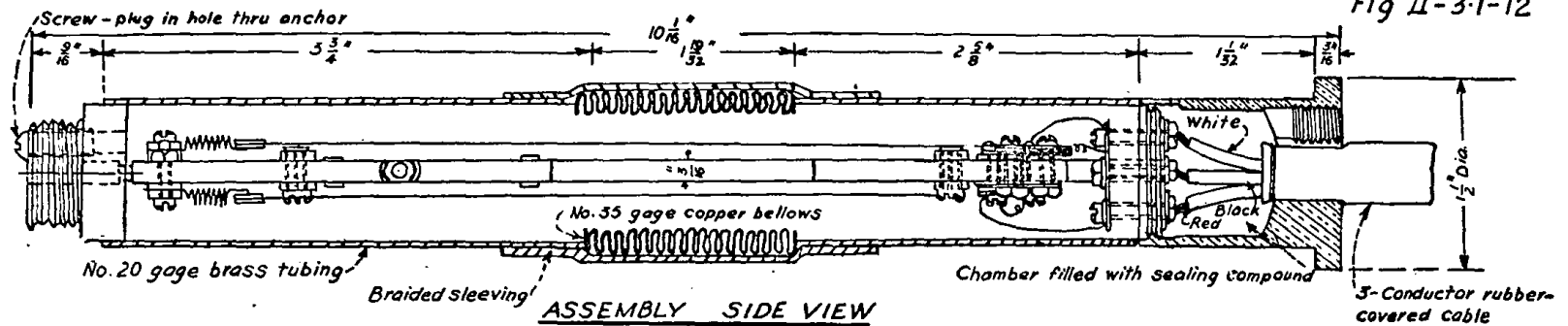
Fig. II 31-11



REVERSE PLUMB LINE

(Deflection measuring equipment and sheet covering not shown)

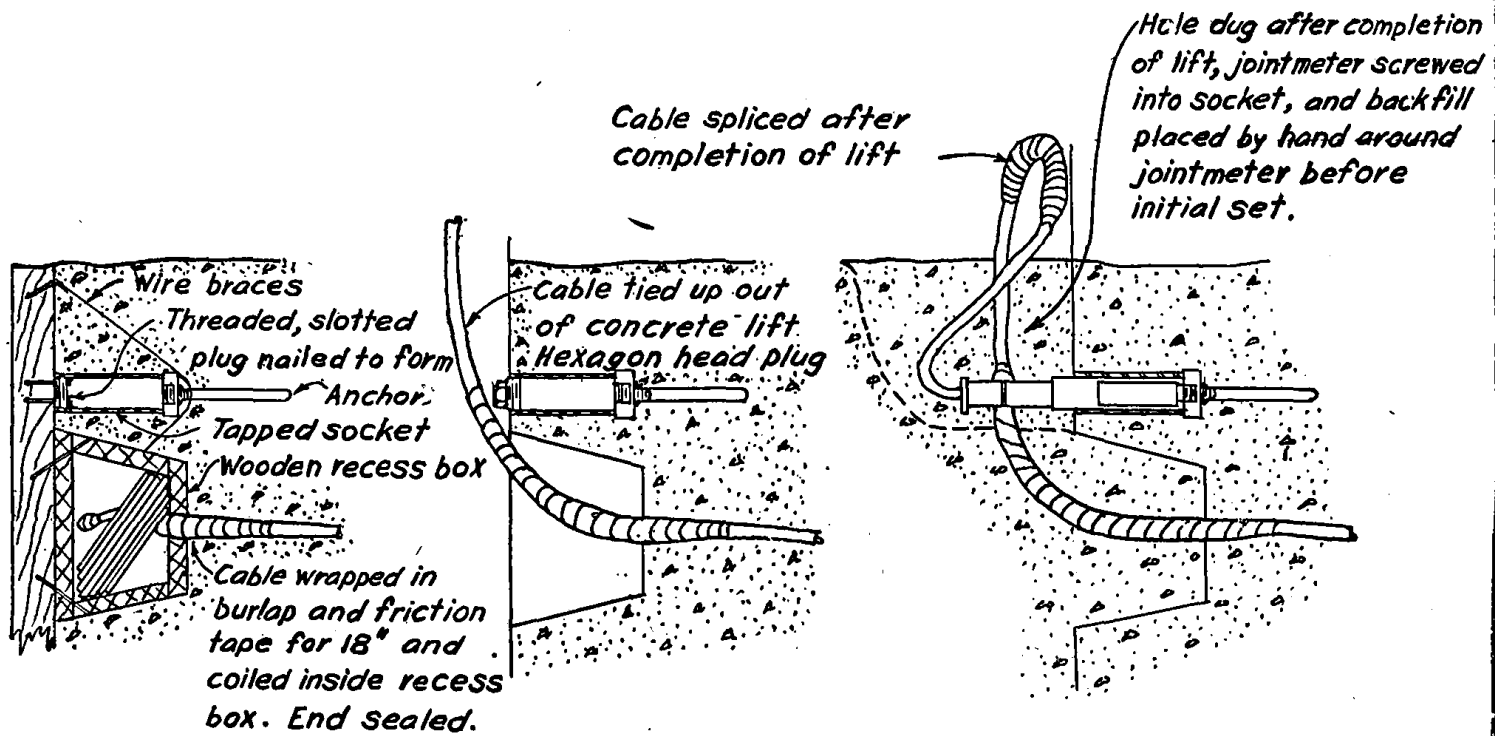
Fig II-3-1-12



NOTES:

- ALL WIRE CONNECTIONS ARE SOLDERED TO BOLTS
- BRASS CASE ENLOSING THE ASSEMBLY IS FILLED WITH CASTOR OIL TO PREVENT CORROSION.
- ALL TERMINALS IN THE ELECTRIC CIRCUIT ARE INSULATED FROM THE SURROUNDING METAL BY TUBING AND MICA WASHERS
- ELASTIC STEEL WIRE IS UNDER AN INITIAL TENSION WHEN ASSEMBLED.
- THIS JOINT METER USED WITH SOCKET AS SHOWN ON DWG. 214-D-10389

MEASUREMENT OF STRUCTURAL BEHAVIOR	
CARLSON ELASTIC WIRE JOINT METER	
DRAWN	
TRACED	
CHECKED	



STEP 1. INSTALLATION IN HIGH BLOCK STEP 2. PREPARATION IN LOW BLOCK STEP 3. JOINTMETER INSTALLATION

JOINTMETER INSTALLATION

FIELD READINGS ON EMBEDDED INSTRUMENTS

Date _____ Time _____ to _____
 Observer _____

TB 6

Inst. No.	Resistance	Ratio
S61		
S62		
S63		
S63A		
S64		
S65		
S66		
S67		
S68		
S69		
S70		
S71		
S72		
St11		
St12		
× J10		
× J11		
× J12		
× J13		
× J14		

TB 7

Inst. No.	Resistance	Ratio
T4		
S73		
S74		
S75		
S76		
S77		
S78		
S79		
S80		
S81		
S82		
S83		
S84		
St13		
St14		
S195		
T5		
T6		
T7		
T8		
T9		
T147		
T148		
× J15		

R9
 R10
 TB8

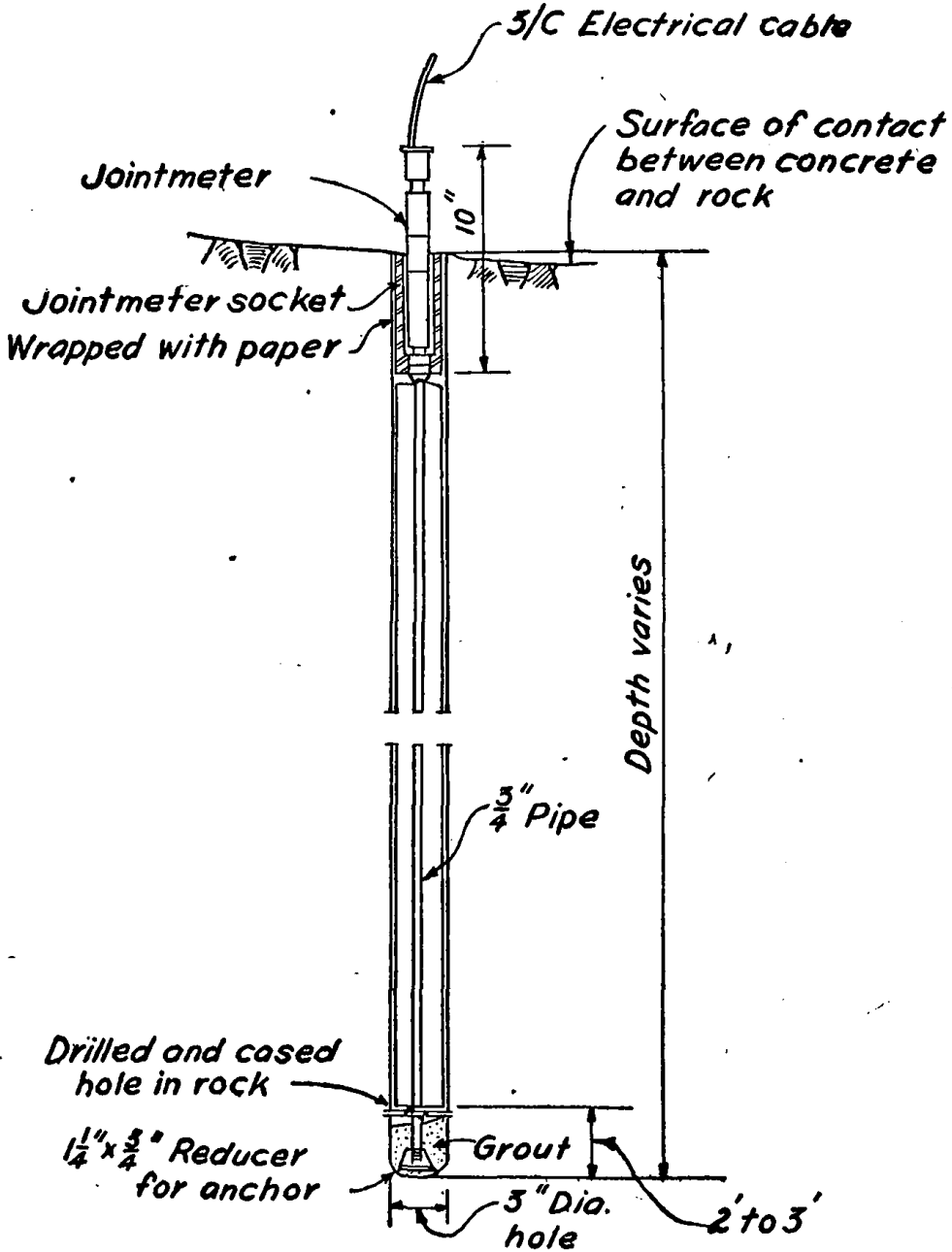
TB 8

Inst. No.	Resistance	Ratio
XJ16		
XJ17		
XJ18		
XJ19		
XJ20		
S85		
S86		
S87		
S87A		
S88		
S89		
S90		
S91		
S92		
S93		
S94		
S95		
S96		
St15		
St16		

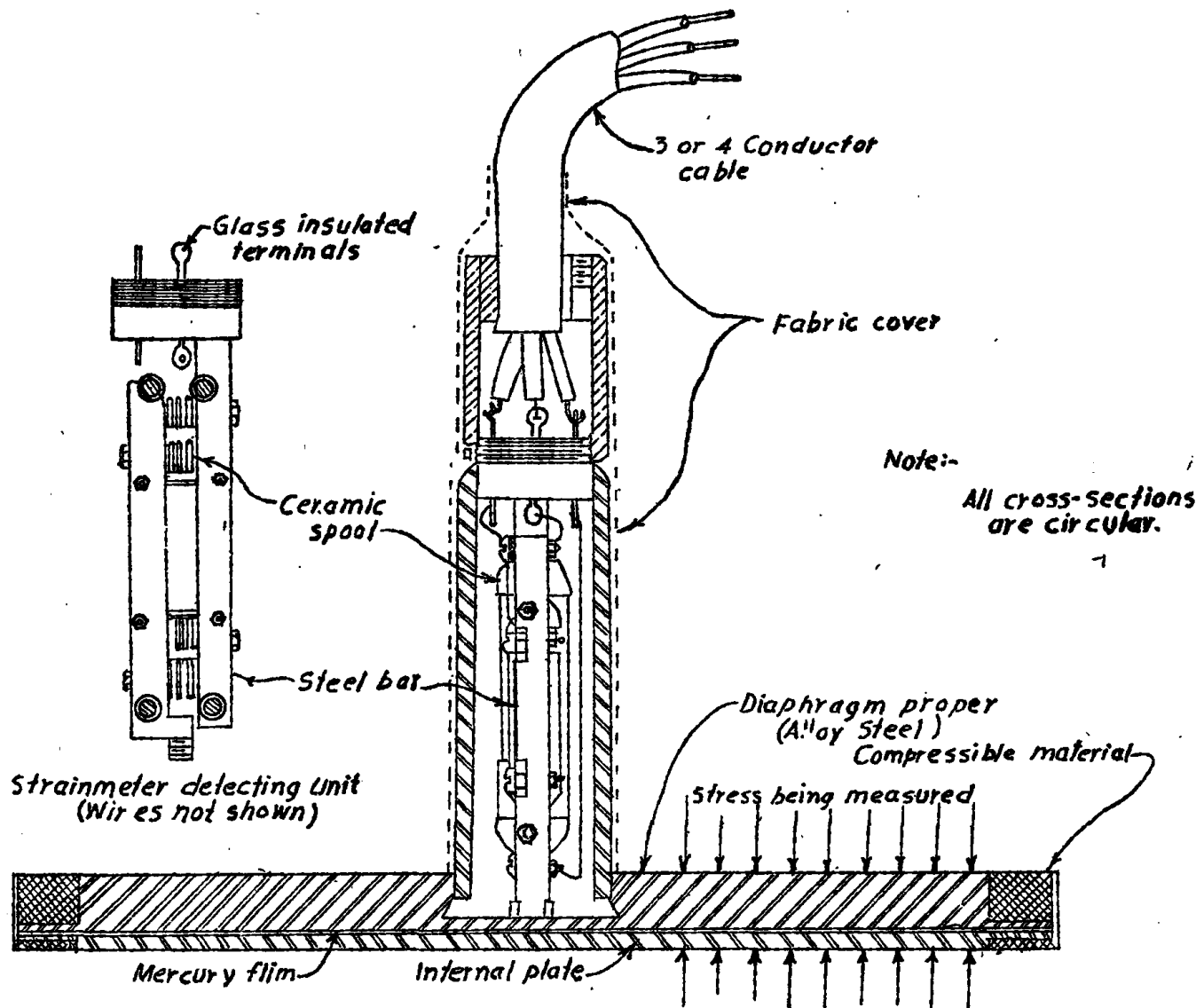
× 3/C. Cables Instruments.

Remarks _____

Fig. II-31-16



TYPICAL DEFORMATION METER



CROSS-SECTION OF STRESS METER
(Not to scale)

Fig. II-3-2-03

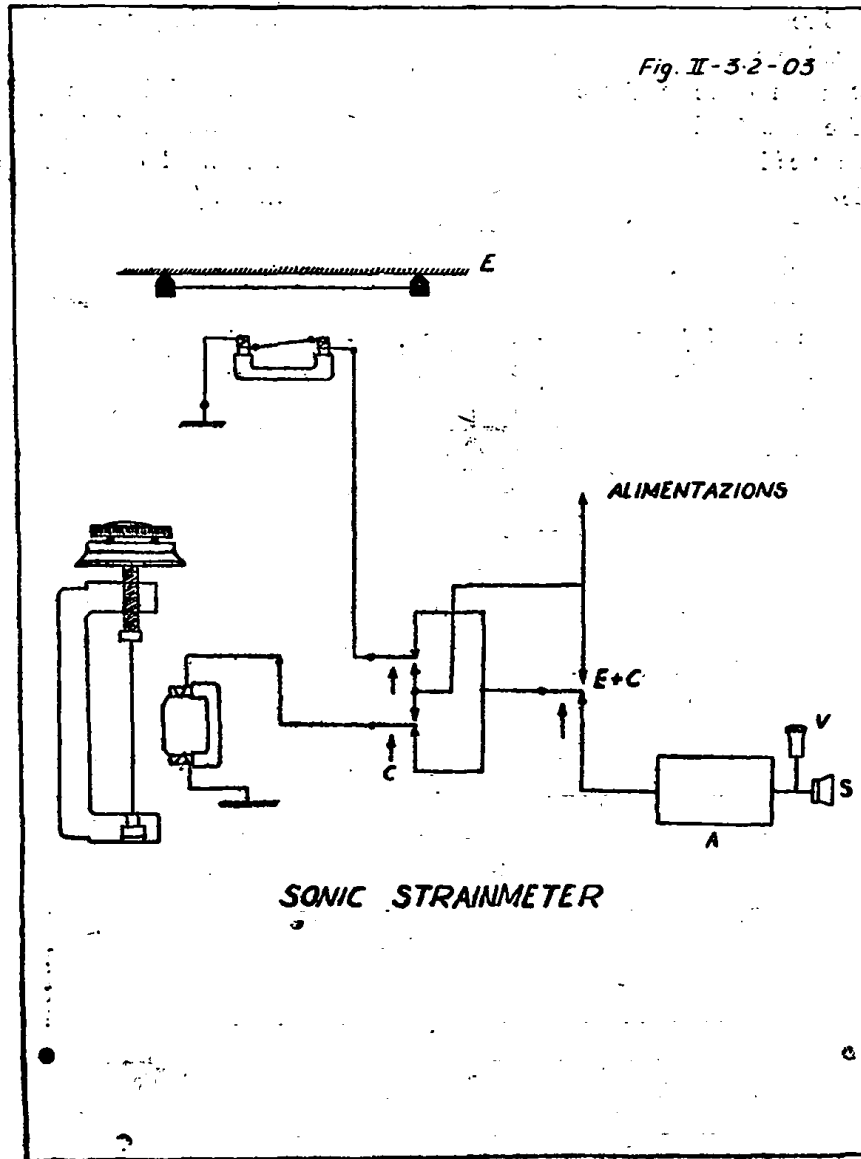
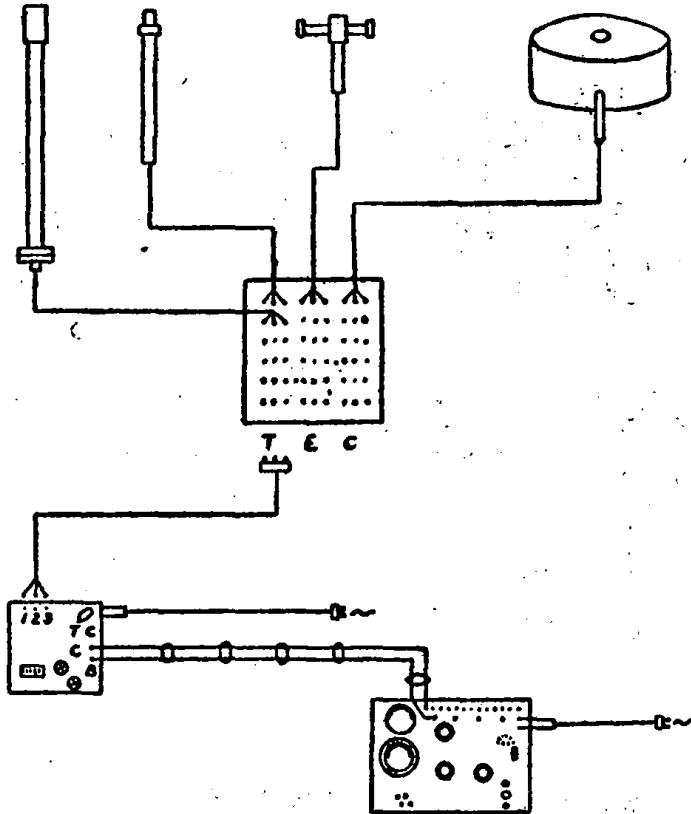
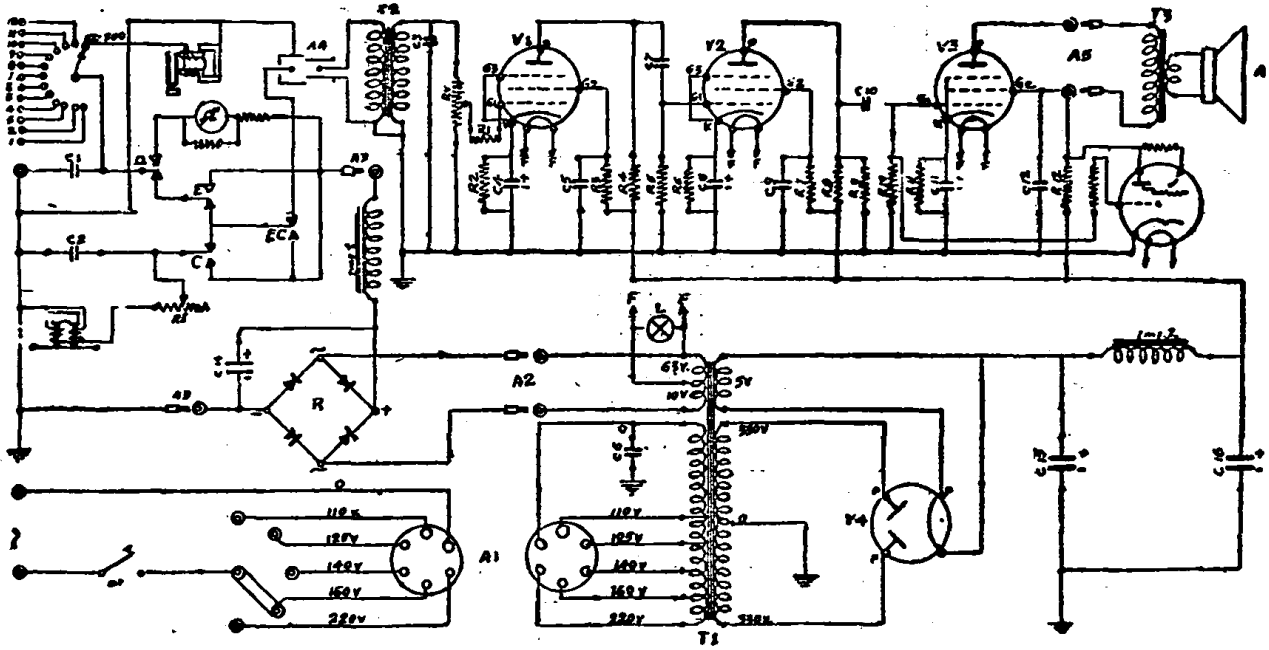


Fig. II-52-04

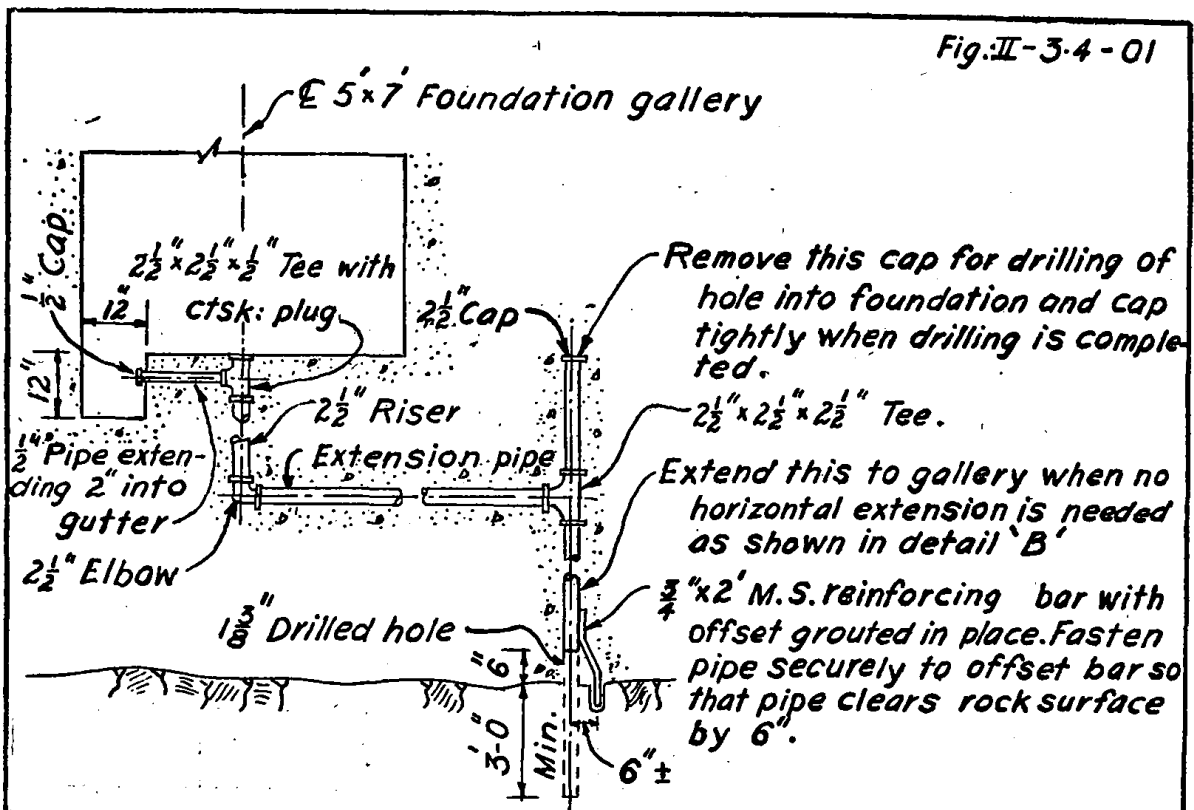


ARRANGEMENT FOR MEASUREMENT
OF TEMPERATURE & STRAIN



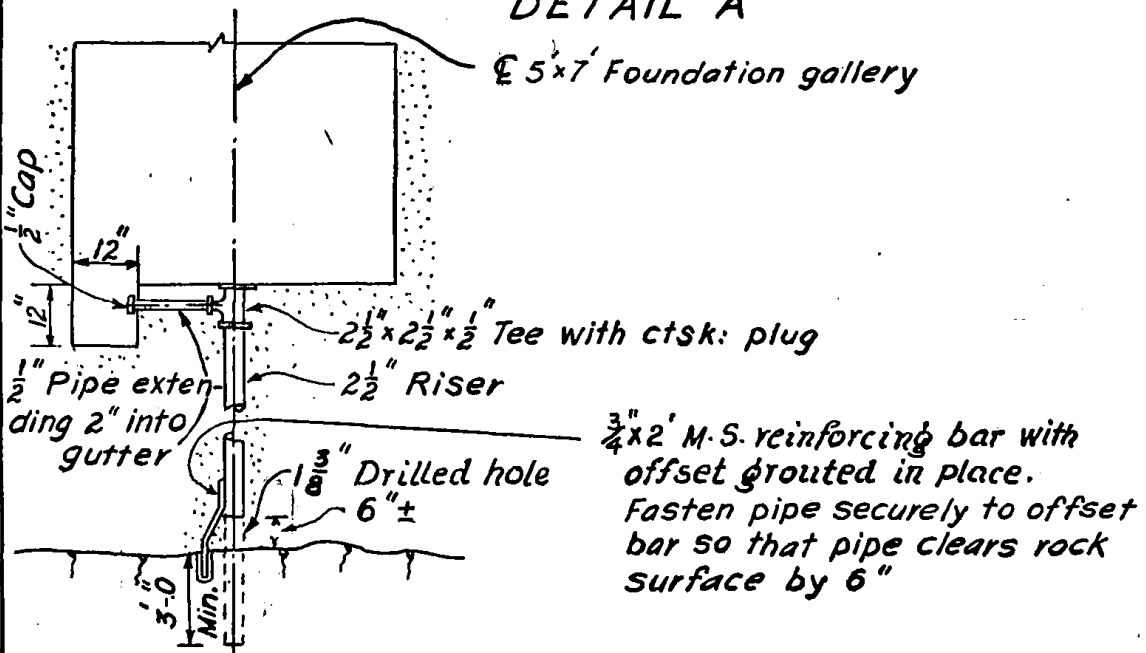
WIRING DIAGRAM

Fig. II-3-2-05



Extension pipe to be used when necessary to connect with riser to gallery to be placed on upgrade of 0.02 towards gallery and at the lowest possible elevation.

DETAIL 'A'

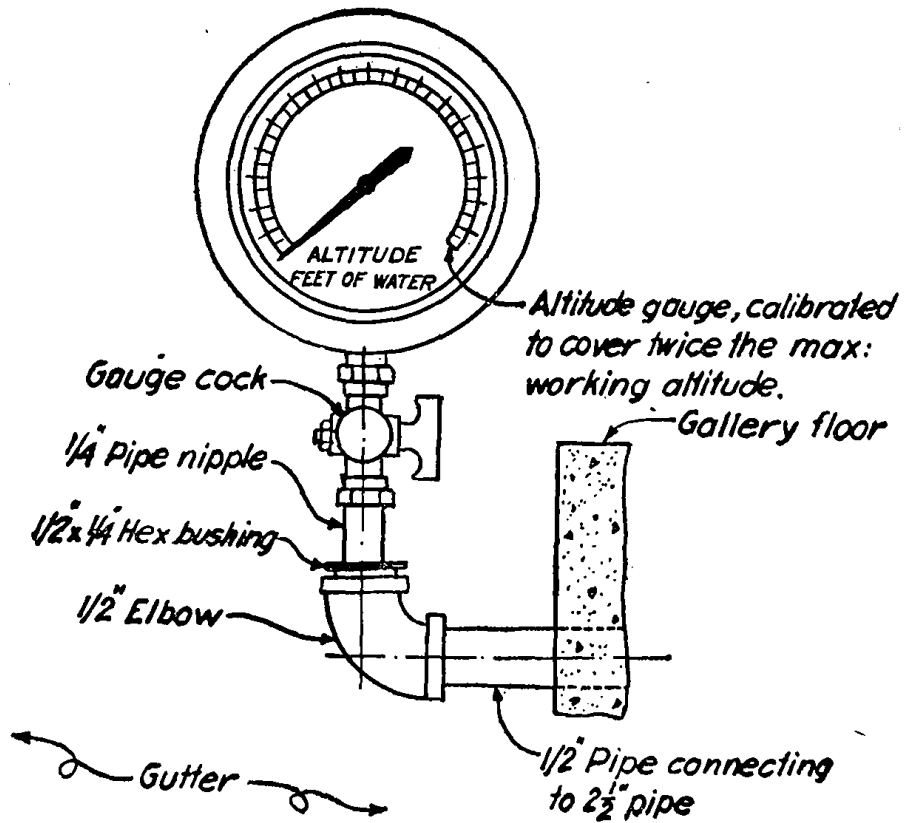


DETAIL (B)

UPLIFT PRESSURE PIPE INSTALLATION

(TYPICAL DETAILS)

Fig. II-3.4-02



TYPICAL ALTITUDE GAUGE INSTALLATION

UPLIFT PRESSURE DATA FORM

Block ----- Sta. -----

Date ----- Time -----

Observer -----

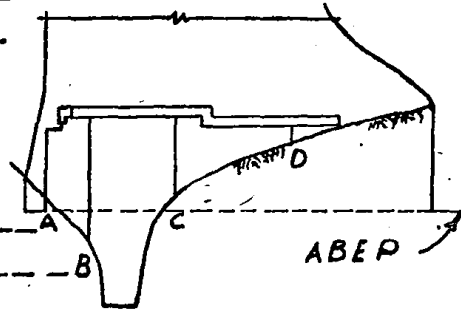
R.W.L. -----

T.W.L. -----

▲ Total width (at line of holes) -----

▲ Av. bottom El. of uplift pipes -----

Net head^{#1} -----



Serial/Column No.

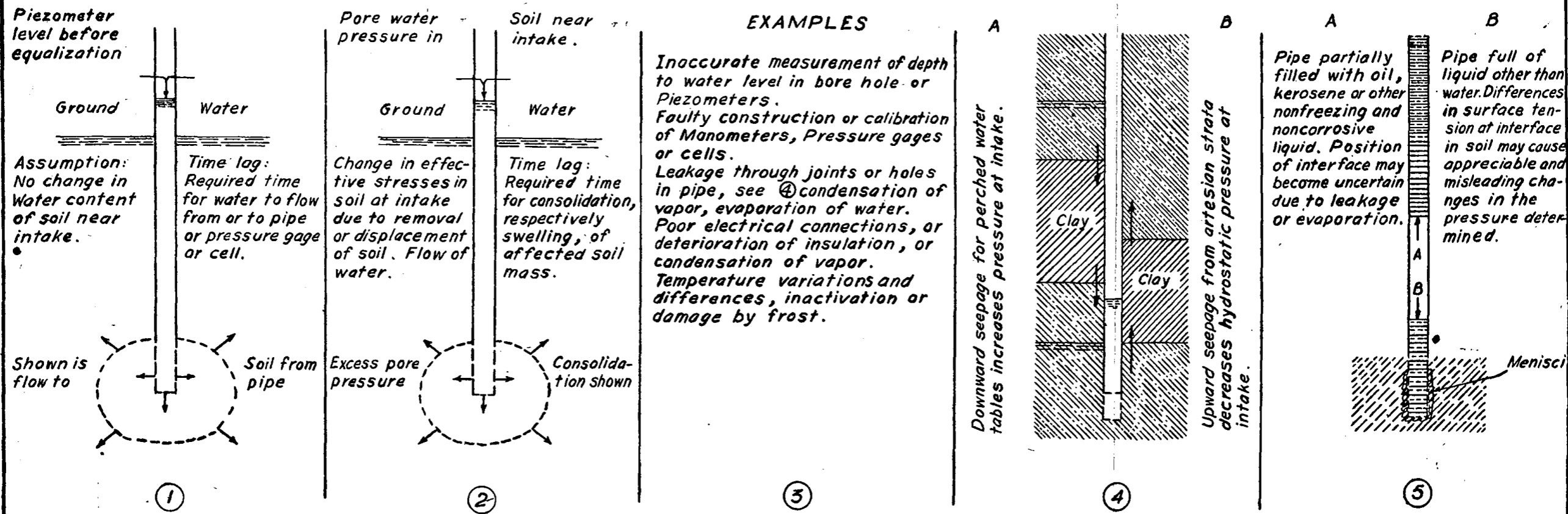
		Pipe A	Pipe B	Pipe C	Pipe D
1	▲ Chainage of Uplift Pipes				
2	▲ Bottom elevation of uplift pipes				
3	▲ Assumed uplift pressures in % age of net head (Criteria)				
4	▲ Distance from u/s end expressed as % age of total width				
5	For pipes not flowing	Δ El. of top of tee			
6		Distance to W.S. in feet			
7		El. of W.S. in pipe = Col. 5 + Col. 6			
8	For pipes flowing	Δ El. of pressure gauge ^{#3}			
9		Pressure head	P. S. I		
10			Ft. of water		
11		Equivalent El. of w.s. in pipe Col. 10 + Col. 8			
12	Discharge in G.P.M./Drops p.m. ^{#2}				
13	Uplift pressure Head in ft. of water above Tail water El. = Col. 7/11 Tail water El.				
14	Uplift pressure in % age of net head = $\frac{\text{Col. 13}}{\text{Net Head}} \times 100$				

*¹ The difference between the reservoir W.S. and the tail water surface elevations, or if there is no tailwater at the line of holes, the net head is the difference between the reservoir water surface elevation and the average bottom elevation of uplift pipes at all uplift holes within a line of holes.

*² State if leakage water contains any foundation material.

*³ El. of pressure gauge is assumed 1.75 ft above gallery floor.

▲ Constitutes permanent data.



HYDROSTATIC TIME LAG

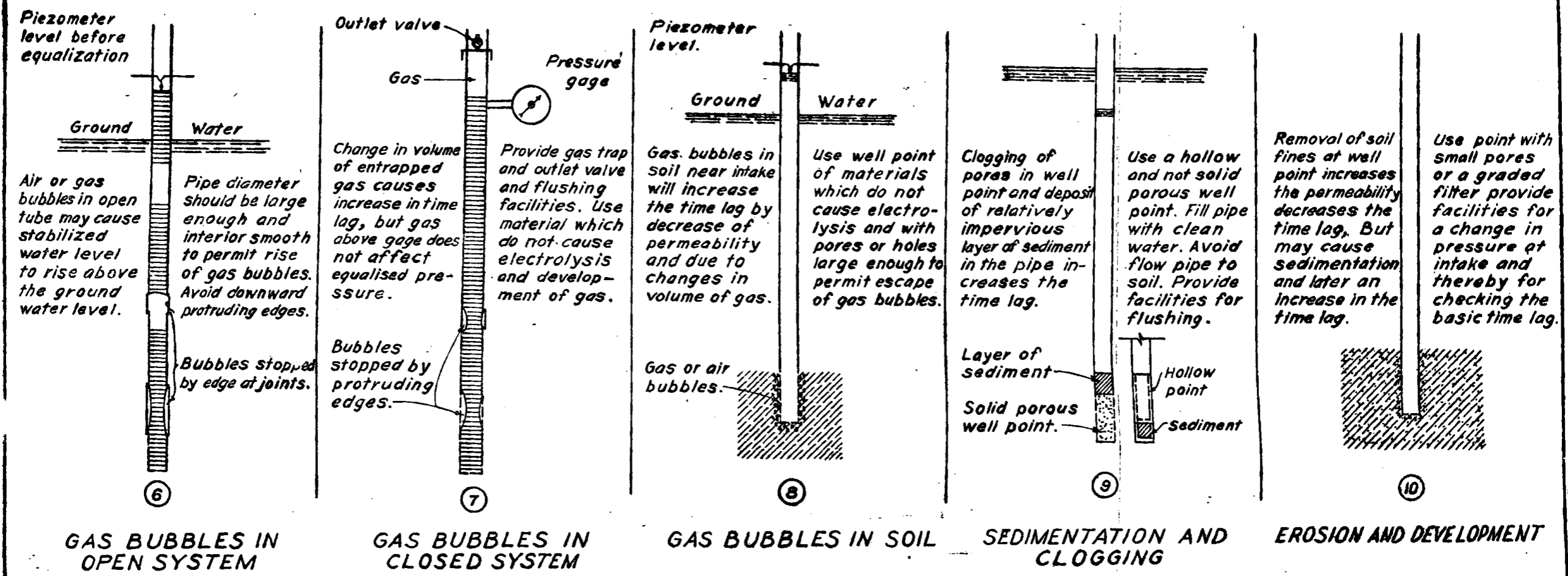
STRESS ADJUSTMENT TIME LAG

GENERAL INSTRUMENT ERRORS

SEEPAGE ALONG CONDUITS

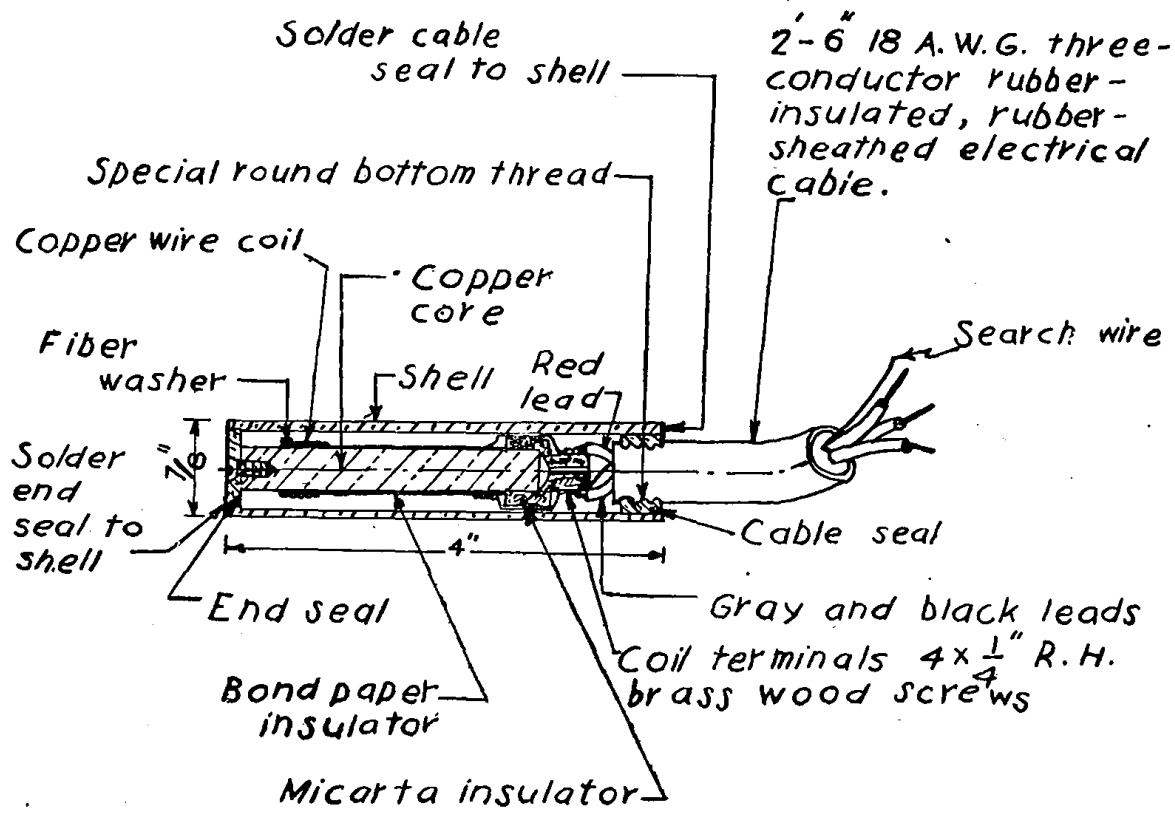
INTERFACE OF LIQUIDS

SOURCES OF ERROR IN DETERMINATION OF GROUND WATER PRESSURE



SOURCES OF ERROR IN DETERMINATION OF GROUND WATER PRESSURE

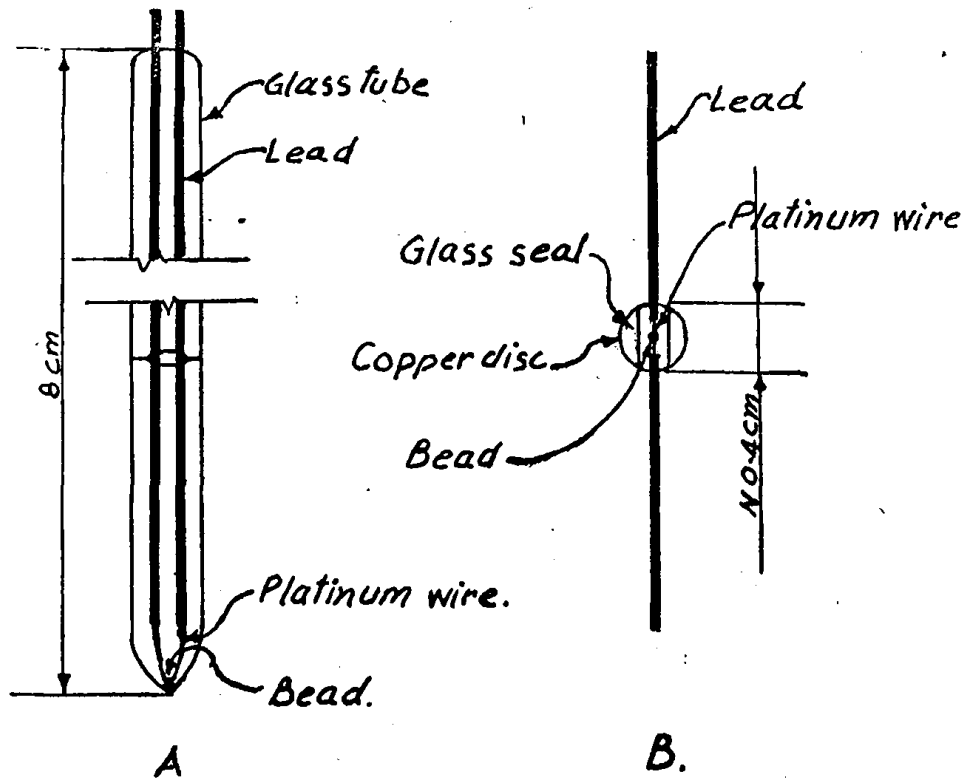
Fig. II-3.5-01



EMBEDDED TYPE RESISTANCE THERMOMETER

NOTES

End seal, cable seal and shell are brass.
Space between thermometer coil and shell is filled with G. E. -227 sealing compound prior to soldering end seal.



Bead thermistors for temperature measurements

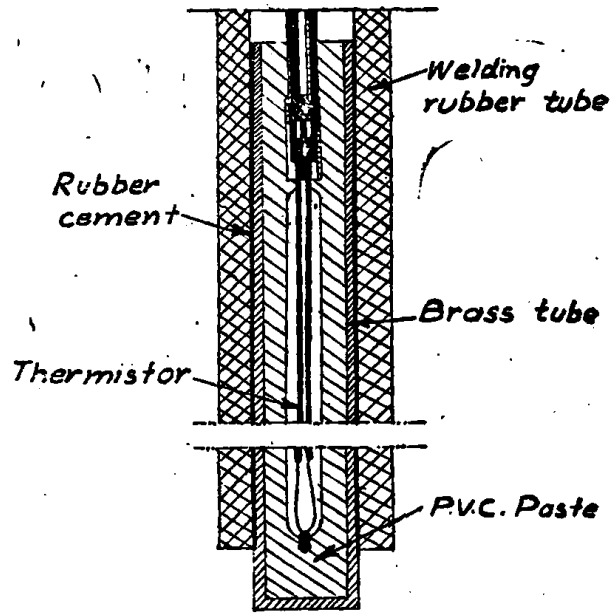


Fig. (a)

Water-proof and shock-proof type of thermometer designed to be embedded in concrete for field uses.

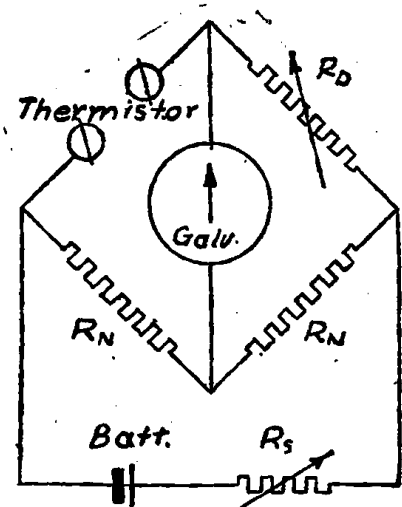


Fig. (b).

Wheatstone bridge for resistance measurements on thermistors.