# STRENGTHENING AND RAISING OF GRAVITY DAMS WITH SPECIAL REFERENCE TO KOYNA DAM

DISSERTATION

*Submitted in partial fulfilment of requirements for M.E. (WRD) of the University of Roorkee*





*By*

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## A C K N O W L E D G E M E N T S

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23/6/75

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# CERTIFICATE

Certified that, the dissertation entitled "STRENGTHENING AND RAISING OF GRAVITY DAMS WITH SPECIAL REFERENCE TO KOYNA DAM", which is being submitted by Er. S.N. Sahasrabudhe in partial fulfilment of the requirements for the award of the Degree of Master of Engineering in Water Resources Development of the University of Roorkee, is a record of the candidate's own work carried out by him under my supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other Degree or Diploma.

This is to further clarify that he has worked for a period of more than nine months between October, 1971 and March, 1975 for preparing this dissertation.

Hari Krishna

Roorkee. (HARI KRISHNA)

Dated :

# C E R T I F I C A T E

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Bombay.

Dated 23rd June 1975

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# SYNOPSIS

Industrialisation and food production have been aptly included on the priority list in a planned all out effort for rapid development on all fronts in India.

Thus increase in Irrigation potential and development of hydroelectric power by full development of water resources in the country is the need of today.

For achieving the above objectives, owing to the unfavourable distribution of rainfall in time and space it has become necessary to store water by construction of dams. However as the availability of suitable construction sites is limited, we will have to consider heightening of some of the existing dams for enhancing their storage capacities. Also with the increased demand for water, some of the old existing structures have been put to an intensive use and therefore it becomes necessary to investigate fully the safety of these structures afresh in addition to exploring the possibility of increasing their storages. Instances can be cited in this connection of Tansa dam (near Bombay), Thokerwadi dam, Shirawata dam and Walwhan dam in Maharashtra State which had to be strengthened. The storage of Tansa dam was increased thrice in the process.

Strengthening of Koyna dam in Maharashtra State can be cited as another major work of strengthening which is now completed.

In order to allow for some forces like uplift not considered in the initial design and to allow for the possibility of occurrence of earthquakes on the Deccan plateau some existing dams in this State, like Bhandardhara dam,Darna dam, Radhanagari dam etc. are also being strengthened.

Thus there is a need for strengthening the existing dams and also a necessity of planning the construction of future works in stages. Strengthening of dams can be done in various ways, a particular method in each case being selected from economic considerations and its suitability for the type of dam to be strengthened.

The aspects of strengthening of gravity dams especially by provision of downstream backing are proposed to be discussed in details in this dissertation. The strengthening of Koyna dam in the State of Maharashtra to which the author belongs is also discussed at length in chapter V as a concrete illustration.

An attempt has been made to bring out all important design and construction problems involved in as many details as possible based on the available literature on this subject.

It is hoped that the discussion presented will at least serve to give some insight into the problems associated with the strengthening of gravity structures, especially those involved in thickening of the existing section.

# CHAPTER I

# NECESSITY FOR RAISING OR STRENGTHENING DAMS

# CHAPTER - I.

## Necessity for Raising or Strengthening Dams :-

1.0 Water is one ofthe five basic necessities for the survial of human civilization. It being so badly needed by man, he could not allow it to go to waste, unutilised, expecially as nature plays the trick of supplyigg it in an irregular fashion. Such unsteady supply of this commodity with respect to time led man to modify nature's landscape with his own creation of obstructions in the form of dams and other structures for storing water at a time when its supply exceeded his requirement and utilising it when the supply fell short of his needs. Thus dam-building is an old art, which has now became a science through technological development.

1.1 In the past dams were constructed to create storages sufficient to meet the need of immediate future. If due to increases in demand for various reasons, more water was required, new storages were created at new suitable dam sites. However, a good dam site, from geological and topographical considerations is a gift of nature, a fact which puts a limitation on the number of separate dams that can be built on a river system. Thus with continuous utilization of favourable sites(including those which can be made suitable by special treatment) a stage is bound to be reached, when no more storage sites are available and yet more quantity of water is required to be stored for further utilization. Under such circumstances, there will be no other option but to raise the existing dams by suitable heights for this purpose.

1.2 Even when some sites are available for constructing new dams, in some cases it can be worthwhile to make a comparative study of two alternatives namely cost of construction of a new dam versus cost of raising of an existing dam per unit quantity of additional storage.

When the latter alternative is cheaper, the same will be preferred from economic considerations. Thus raising of the existing dams can be an alternative solution to the creation of seperate additional storages in such cases.

1.3 Thus exhaustive utilisation of water resources needed to meet the ever growing demand for water is one of the main reasons for raising of dams. Strengthening of dams is generally needed in most of the cases when raising is to be done. It is on the other hand also needed to make a dam stable, which is considered to be rather insecure due to reasons which can be several. In the latter case, it can be in the form of a repair work, while in the former it is no doubt an original work.

1.4 To clearly differentiate between raising and strengthening, we can say that raising is that additional part which is constructed above the top of the existing dam to increase storage capacity while strengthening is that additional part constructed below the top of the existing dam in connection with raising or repair.

1.5 Some of the reasons (1) which have necessitated raising or strengthening of dams in the world could be summarised as below :

(i) <u>Increase in population & Economic Development</u> :

One of the principal reasons for raising of dams has been ever growing demand for water due to increases in population as well as rapid pace of industrial development. Growth of population leads to greater demands for water supplies for domestic use as well as growing more food. The demand for increase in food production results in increase in irrigation requirements thus increasing the demand for irrigation water. In under-developed countries, where development plans for rapid progress are pursued with vigour, this demand is bound to grow at a tremendous rate, which means more exhaustive utilisation of water resources. This stresses the importance of this subject in the present times.

Examples for raisings done on this account are many. To cite few of them, we can mention Asswan Dam in Egypt which was raised twice, due to increases in irrigation demand and Tansa dam which was raised thrice to meet water supply needs of Bombay in Maharashtra State.

(ii) Reapprisal of data:

(a) At the time of construction of dams, the hydrological records available may not have been sufficiently exhaustive for accurately assessing water potential of the catchment. Thus revision of hydrological studies based on available additional hydrological records may lead to consideration of increasing the storage capacity of a dam to minimise wastage.

Odomari dam in Japan was raised by 10.5 metres as it was of inadequate capacity causing wastage of water by overflow.

(b) Data on silt load carried by rivers is not generally available at the time of construction. If subsequent to the construction of dam, it is realised from data collected since then that heavy inflow of silt may be expected, it may be found necessary to increase the storage space allocated for silt-deposition in order to maintain the useful life of the reservoir. This can be done by providing extra storage capacity by heightening the dam by the required amount.

Jalaput dam in the Rajasthan State of India is such an example. Thus originally estimated dead storage capacity of 7 M.cm. was subsequently revised to 77 M.cm. which needed a raising by 3 metres.

The Guayabal Dam in Puerto Rico is another example. The reservoir silted up within 31 years to the point where its capacity was reduced to 40% of the original volume, which led to reduction of irrigated area. Raising was done to increase the pond-level by 4.8 metres to restore the original live capacity.

(c) In some instances, the need to allow for some forces, which could not be foreseen at the time of initial design is realised at a later date, necessitating thereby strengthening of such structures.

Koyna Dam in Maharashtra State, India, is one such example. The dam has been constructed in the Deccan Plateu which was considered to be an aseismic zone, until the area near Koyna dam experienced a strong earthquake in 1967. This led to reconsideration of design criteria as regards magnitude of earthquake acceleration to be allowed for in the computations and consequently to the necessity of subsequent strengthening.

(iii) Evolution of Design-Criteria :

There have been cases in which the actual loads acting on the dam exceeded the loads considered in its design. Several examples can be cited of masonry or concrete dams constructed in the past where no or inadequate uplift allowance was provided for in the design and thus the dam sections when analysed on the basis of present design criteria showed tensile stresses on their upstream faces which are considered undesirable in the design of gravity dams. Such dams require strengthening which can be done by addition of weight to correct the stress conditions. In such cases, when strengthening is to be done, heightening can also be considered simultaneously as an incidentally possible proposition.

Examples-Cheurfas dam in Algeria, Tansa and Shirwata dams in Maharashtra State in India, Barker dam in America.

(iv) Deterioration of materials

Repairs have been found necessary in several cases to arrest deterioration of mortar or concrete at early stages, which otherwise at a later stage may result in disintegration and high rates of leakages. The reasons for such decay may be frost action, severe temperature conditions, acidic waters, bad workmanship and many other factors.

Asswan dam in Egypt and Mosvann dam in Norway had to be strengthened to reduce leakages and arrest deterioration due to acidic waters. Ringedal Dam in Norway which was affected due to severe temperature conditions ($85^{\circ}$ F to $0^{\circ}$ F) and acidic waters (PH value $\approx$5.9) had to be strengthened.

Odomari dam in Japan suffered from weathering action on upstream and downstream faces. Concrete in the dam was also not of a good quality. This and other considerations led to its strengthening and raising.

(v) Better Efficiency in Hydro-electric Schemes

Fluctuations in the reservoir level require elasticity in generators to maintain efficiency. The fluctuations are high, when the stored water is mainly used for irrigation purposes. Heightening of a dam places the volume affected by regulation on a higher position because of the corresponding raising of the minimum drawdown level which because of bigger surface area at higher levels gives smaller fluctuations in level under the same regulation. Thus a greater useful head as well as a reasonably constant head could be obtained. This sort of heightening is thus more applicable for hydro-electric utilization of reservoirs created basically for regulation with irrigation.

as the purpose.

(vi) Advances in Technology

Technological advances in design and construction methods may justify heightening of dams in some cases. For instance, advances in the methods of analysis of hydrological data may sometimes indicate the need for increased flood handling provisions. Economic considerations may often involve a combination of increased spillway capacity and addition of temporary surcharge storages achieved by heightening of dams.

Examples - Alamogordo dam in New Mexico.

1.6 Information about gravity dams heightened in the present century is given in Table No.I.

1.7 Thus heightening and strengthening of dams may be necessary in several cases from considerations of safety or better utilization of water resources in the most economical manner. In the cases of countries like India, where water resources play an important role in the economy of the country, significance of this aspect is still greater.

Table -1

List of gravity Dams raised in height

Notations:- IR - Irrigation
P - Power
WS - Water Supply.

| River | Purpose of Storage | Type of Construction. | Year of Constn. | Year of raising height | Initial height h (metre) | Increase in height Dh(metre) | Initial Storage $Mm^3$ | Additional storage due to raising $Mm^3$ | $\frac{Dh}{h} \times 100$ |
|---|---|---|---|---|---|---|---|---|---|
| 4. | 5. | 6. | 7. | 8. | 9. | 10. | 11. | 12. | 13 |
| ...arrance ...ande | IR | Masonry | 1384 | 1586 | 14 | 7 | - | - | 50 |
| - | WS | Concrete | 1917 | 1924<br>1939 | 32 | 2.5<br>9.45 | - | -<br>2.27 | 7.82<br>27.6 |
| ...uela | - | Concrete | - | 1926 | 24 | 3 | - | - | 12.5 |
| ...ile | IR, P | Masonry | 1902 | 1912<br>1933 | 30<br>- | 5<br>15 | 1000<br>- | 1500<br>2500 | 16.7<br>42.9 |
| ...von | WS | Concrete | 1956 | - | 29 | 4 | 1.4 | 0.70 | 13.8 |
| - | - | Concrete | 1942 | - | 29 | 4 | 15 | 2.2 | - |
| ...ajo | P | Concrete | - | - | 23 | 13 | - | - | 56.5 |
| - | WS | Masonry | - | - | - | 18 | 18 | - | 100 |
| Murrumbidgee | IR, WS, P | Concrete | 1913 | 1945 | 75.5 | 5.2 | 810 | 230 | 6.9 |
| - | - | Concrete | - | 1956 | 26 | 6 | - | - | 23.1 |
| ...ekerra | IR | Masonry | 1891 | 1930 | 40 | 3 | 15 | - | 8 |
| Turon | P | Concrete | 1921 | 1948 | 80 | 2 | 80 | - | 2.5 |
| Flumen | IR | Masonry | - | - | 11 | 10<br>10 | -<br>- | -<br>- | 91<br>47.7 |
| Dix | WS | Masonry | - | 1904 | 5 | 1 | - | - | 20 |
| Navia | P | Concrete | 1934 | - | 94 | 4 | 102 | 16 | 4.25 |
| - | - | Concrete | 1904 | - | 44 | 10 | 2-3 | 10.3 | 23 |
| - | - | Masonry | 1910 | - | 10 | 6 | 1 | 6 | 60 |
| ...al Des Dix | P | Concrete | - | - | 182 | 42<br>30<br>30 | 100<br>-<br>- | 94<br>91<br>115 | 23<br>13.5<br>12. |
| ...uadalm...llato | - | Concrete | - | 1952 | 56 | 8 | 110 | ~~11~~52 | 14.3 |

Table -1

| River | Purpose of Storage | Type of Constru-ction. | Year of Constn. | Year of raising height. | Initial height-h (metre) | Increase in height Dh(metre) | Initial Storage $Mm^3$ | Addition-al sto-rage due to raising $Mm^3$ | $\frac{Dh}{h}$ |
|---|---|---|---|---|---|---|---|---|---|
| 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 |
| ergong | - | - | 1873 | - | 36 | 6 | - | - | 17 |
| Irati | - | - | - | - | 15 | 12 | - | - | 75 |
| | | | | | | 7 | - | - | 26 |
| | | | | | | 6 | | | 17.5 |
| | | | | | | 9 | - | - | 22.5 |
| Machkund | P | Masonry | - | - | 52 | 3 | 724 | - | 5.8 |
| - | P | Concrete | 1908 | 1948 | 32 | 28 | 182 | 820 | 87.5 |
| Colorado | - | Concrete | - | 1939 | 60 | 23 | - | - | 38.2 |
| - | - | Concrete | 1940 | - | 16 | 4 | 0.8 | 0.3 | 25 |
| - | - | Concrete | - | - | 36 | 6 | - | - | 16.7 |
| Helena | WS,IR | Concrete | 1902 | 1948 | 30 | 10 | 21 | 48 | 33 |
| Ota | P | Concrete | 1935 | - | 60 | 10 | 13 | 13 | 16.7 |
| Hetch Hetechy | WS | Concrete | 1923 | 1938 | 105 | 26 | - | - | 24.8 |
| - | P | Masonry | 1924 | 1924 | 38 | 13 | 6.7 | 9.9 | 34 |
| Lozoya | WS | Concrete | - | 1935 | 28 | 36 | - | - | 128.50 |
| - | - | Concrete | - | 1913 | 11 | 4 | - | - | 36.30 |
| - | P | Masonry | 1920 | - | 39 | 3 | 186 | - | 7.2 |
| - | WS | Concrete | - | - | 30 | 2 | - | - | 6.7 |
| - | WS | Masonry | 1892 | 1914<br>1951 | 36 | 3<br>2 | - | - | 8.3<br>5.1 |

# CHAPTER II

# METHODS OF HEIGHTENING OF DAMS.

# CHAPTER - II

## METHODS OF HEIGHTENING OF DAMS

### 2.0 General

Broad classification of the methods of heightening of dams can be done into two groups :

1. Direct heightening,

& 2. Strengthened heightening.

The second catagory can again be further subdivided into methods which use different techniques for strengthening. These are given in Table No.2 enclosed at the end. These techniques are proposed to be discussed in brief in this chapter. Sketches illustrating these methods (3) are given in the figures 2.1 & 2.2.

### 2.1 Direct Heightening

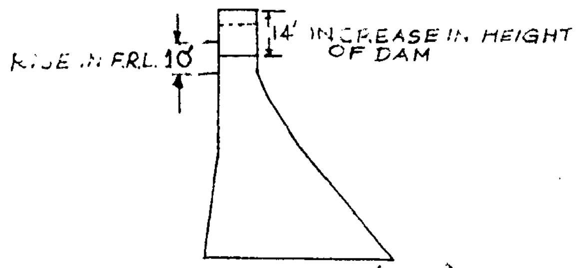
This refers to the addition of a new section on top of the existing structure for the purpose of creating extra storage capacity without requiring any strengthening measures for securing the stability of the existing structure for withstanding the increased forces.

Thus in case of direct heightening, the section of strengthening is not necessary. This of course is possible only if the existing structure has already a sufficient margin of safety to resist the increased stresses caused by increases in the loads acting on the structure.

2.1.1 If the reinforcing structure on the downstream can be avoided, the same results in appreciable savings in cost. In some cases, this reinforcing structure can possibly be avoided by resorting to the following techniques.

(a) The dam could be heightened to a level higher than that required to create the necessary increase in storage, if this additional weight could be useful in providing the necessary stabilizing weight to create safe stress conditions in the structure.

Examples of such heightenings are Ennepe Dam in Germany, and Jalaput dam on Machkund river in India (2). Please refer figure given below:




JALAPUT DAM (INDIA)

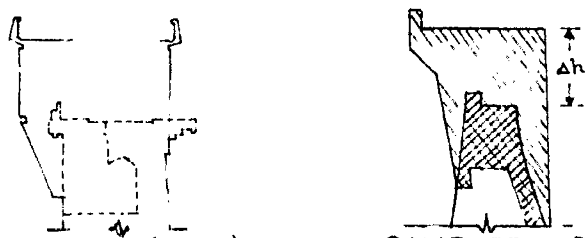
(b) The required stabilizing weight could be obtained by overhanging the new section on the upstream face of the dam.

Examples of such construction are

i) Chorro dam in Spain, and

ii) Doiras dam in Spain (19).

Please refer figure given below:




DOIRAS DAM (SPAIN) CAMPOFRIO DAM

NOTE :- Δh = HEIGHT OF RAISING

(c) Reduction of the external loading can also be attempted wherever feasible. Thus in some cases, the forces of uplift were appreciably reduced to permit direct heightening by emptying the lake and building levy facing on the upstream side of the dam. This was done in the case of Ringedal dam in Norway. In the case of upstream strengthening of Mosvann dam in Norway, elimination of uplift was achieved by building an inclined R.C.C. slab on the upstream side after emptying the lake. Figure 2.2.2 illustrates this arrangement.

(d) In designing dams with overfall spillway, temporary surcharge head produced by the overflowing water is considered. Design reserve for this transient load could be employed for permanent raising of the storage level by the provision of mobile gates.

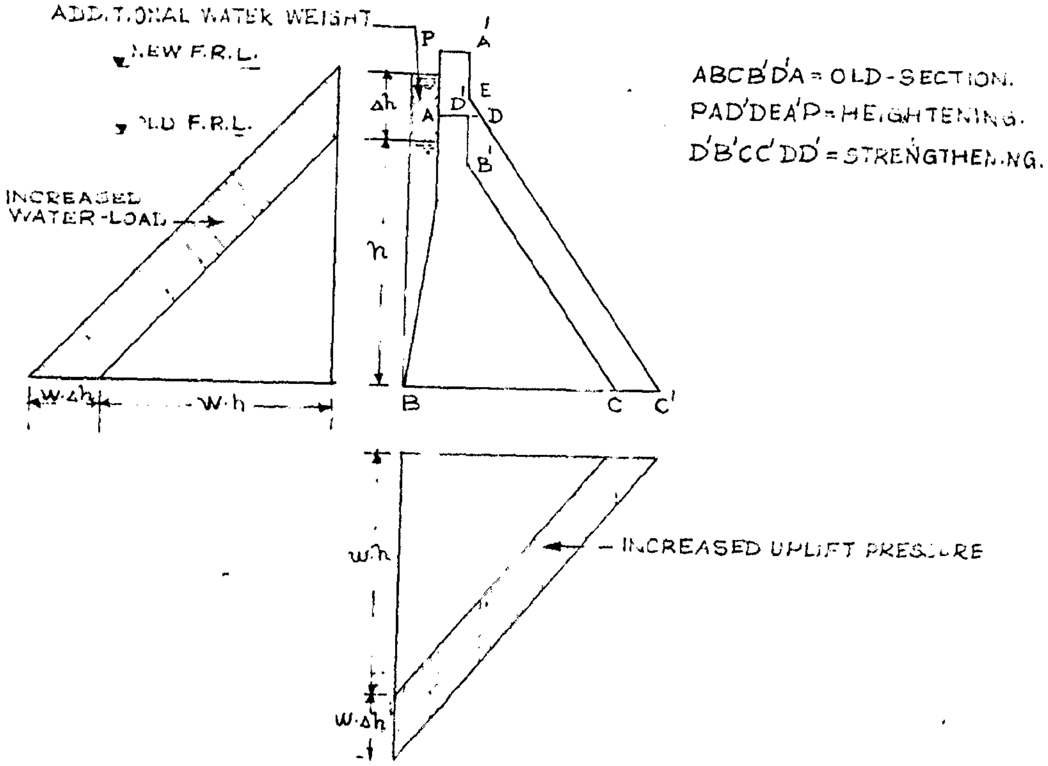
Example - Bristol dam in U.S.A.

(e) In some cases, direct heightening may be possible, if some relaxations in the design assumptions could be permissible in a particular case. For instance we have standardized on the considerations of uplift pressures in the design. On the basis of actual observations of uplift pressures under a particular dam to be heightened, it might be just possible that assumption of existence of reduced pressures may be permissible in design in which case possible adequacy of direct heightening could be thought of.

2.2 Strengthened Heightening

When the old section requires reinforcement to resist safely the increased loading, it becomes a case of strengthened heightening. As pointed out earlier in Chapter I, strengthening is also be needed in the case of dams affected by severe weathering conditions as well as for those structures which are subjected to external forces of magnitudes greater than those allowed for in the original design, even when such structures are not to be raised.

The figure given below shows how the old section will be subjected to increased forces of horizontal water pressure, uplift pressures, additional weights of water on the upstream face as well as the weight of the new section added at the top in case strengthening is to be done from the downstream side.




2.1.1 <u>Conventional Methods of strengthening</u> :

Under the conventional methods of strengthening gravity dams, the strengthening may be done on the upstream side or the downstream side. As a third alternative, the existing dam could be made a core by its envelopment on both sides by earthfill.

a) <u>Upstream strengthening by use of seperate R.C.C. slab on the upstream side</u> :

In some cases, due to infeior quality of materials used in the construction of the existing structure or due to weathering action by the attack of acidic waters in storage or severity of temperature conditions in the region, strengthening by only thickening the dam section may not be considered desirable, as securing of good bond between the two works may not be relied upon (5). In such cases, an upstream inclined slab could be added. Please refer to figure 2.2.2. Such an arrangement can provide the following advantages:

i) Elimination of uplift pressures below the existing dam structure ;

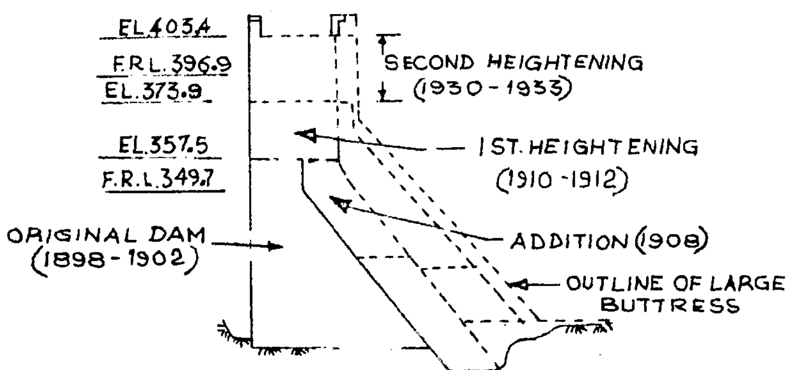
ii) Prevention of contact of the materials of the existing dam with the aggressive storage - water thus increasing the durability of the existing structure;

iii) Making the old structure free of a part of the water load as some water load may get transferred directly to the foundation rock.

Adoption of such a method, however, requires keeping the reservoir empty during the construction period thus involving loss of storage water. This factor must therefore be considered in evaluating comparative economy of this method.

b)Provision of masonry backing on the downstream side:

Strengthening by thickening on the downstream side can be done by provision of a concrete or stone masonry facing either continuous or in the form of buttresses resting on the downstream face of the existing section, not by bonding but through frictional contact (21). Second heightening of Asswan dam is an excellent example of this type of construction (11). The same has been schematically shown in the figure given below :-




ASSWAN DAM (EGYPT)

This method is not generally preferable due to the danger of entrance of water through joints, which can cause dangerous hydrostatic uplift pressures challenging stability of the structure itself in case the drainage system fails to give adequate relief. There is also likelihood of concentration of local stresses occuring due to varying degrees of frictional contact.

(c) <u>Provision of bonded section</u> :

This refers to the provision of a concrete or stone masonry facing either continuous or in the form of buttresses well bonded to the existing structure (16) either on the upstream or on the downstream to form a monolithic structure. The modified dam will then have nearly the same profile as that of one which would be necessary for a new dam to resist all the forces including the extra forces now expected to act after raising.

Examples are - i) First raising of Asswan dam in Egypt,

ii) Strengthening of Lages dam in Brazil,

iii) Mullardoch dam in Scotland,

iv) Koyna dam in India,

& v) Marshall Ford Dam in U.S.A. - ( Upstream strengthening).

This bonding may be done either simultaneously as the construction of the added section proceeds or the same may be effected at a later date under appropriate conditions. The latter procedure is considered more preferable as discussed later.

(d) <u>Earth Backing</u> : This is yet another method of strengthening on the downstream side. Here, instead of thickening the dam by the construction material used originally, an earthen section is added on the downstream of the structure for support. This can be considered as the simplest solution if conditions as well as economic considerations permit its use.

Examples are i)Khadakwasla dam in Maharashtra State, India;
& ii)Walwhan dam in Maharashtra State, India.

In strengthening by earth-backing, estimation of the correct value of the coefficient of earth pressure is very difficult due to which correct estimation of the stress-picture is rendered difficult. Secondly the earth backing which is necessary to produce compressive stresses on the upstream face of the dam in lake-full condition, may cause tensile stresses on the downstream face of the dam in the lake-empty condition. Therefore it is not so suitable when the variation in the reservoir water level is high. It is also not a suitable method when the dam is to be strengthened against earthquake forces. This is so as during earthquake tremors, the gravity dam and the earth backing may vibrate with a phase difference and loose contact in partial height near the top of the earth backing resulting in loss of support. The actual earth pressures acting on the dam during the oscillations fall below their values under static condition thus causing a decrease in the support from the downstream side during the earthquake tremors (47). Lastly, in case of dams having several undersluices, provision of earth-backing cannot be considered (11). Due to these limitations of this method, it appears to have been adopted in a few cases only.

(e) <u>Earth Envelopment of dam section</u>:

This may be a possible solution in some cases, for instance as in the case of Bever dam in the U.S.A.(1).

2.2.2 <u>Use of Prestressed Cables</u>:

This is done by drilling holes through the body of the dam near its upstream face as far as possible. These holes are drilled upto a sufficient depth into the foundation rock below the foundation level of the original structure. High tensile steel wires or cables are inserted into these holes and anchored in the reamed bottom portions of the holes by grouting. They can then be sufficiently stressed to induce the desired compressive stresses along its line in the structure to increase its stability.

This method takes the advantage of good foundation conditions available at the dam site by bringing into action the weight of a part of the foundation for the structural balance of the heightened dam, and can thus be used only where such conditions exist.

Some of the limitations of this method are :

(1) Loss of prestress with time due to yielding of the steel as well as plastic flow of concrete in the dam under the action of localised and sustained loading.

(2) Possibility of corrossion of the steel wires.

(3) Attainment of unyielding anchorage is a difficult job.

So also as large forces are required to be sustained in the foundation, a limitation is imposed on the height of the dams which may be strengthened by this method. Good rock foundations and sound construction material in the body of the dam are other pre-requisites for the use of this method. The use of prestressed cables near the upstream face will not be a satisfactory solution when strengthening of the dam is to be done mainly to resist forces acting during an earthquake, when the variation of the lake level in the season is high. This is because, if the earthquake forces act from downstream towards upstream when the lake-level is low, large tensile stresses will develop on the downstream face.

This method may however be considered preferable for relatively low dams where installation of prestressed cables will be simple and where no possible hazards to life and property are involved in case of failure (7). This will continue to be so until we get confidence about its long term dependability from the experience of such already executed works.

Examples are i) Cherufas dam in Algeria,
ii) Tansa dam in India;
& iii) Bhandardara dam in India.

2.2.3 <u>Use of Hydraulic Jacks</u> :

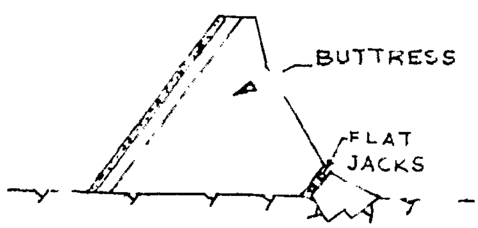
Use of hydraulic flat jacks can be made on the downstream side to apply thrust from downstream to upstream direction to create stable conditions. Thus use of cables, drilling etc. is avoided. The thrust can be applied wherever we like by putting in the flat jacks at the back thus affording flexibility. Although thrusts as high as 10,000 to 20,000 tons can be applied, owing to the creep of the concrete of the dam, the force is reduced significantly in course of time. The method is better suited for buttress structures (6).

Examples : (1) Beni-Bahdel, a multiple arch dam in Algeria, which was raised by 7.32 m.

(2) Djen-Djen dam in Algeria, also a multiple-arch dam.

(3) Memjil dam in Iran, a buttress type dam.

The arrangement is indicated in the figure given below:




BUTTRESS DAM

## 2.3 Selection of the method

The method of strengthening to be adopted in a particular case has to be decided taking into account the topographical, climatological and geological conditions at the dam site, type of the dam to be strengthened, condition of the existing dam, means available for undertaking the work, construction difficulties involved, aesthetic considerations and comparative economy.

In this paper, it is proposed to go into the design and construction aspects in respect of strengthening of gravity dams by provision of concrete or masonry backing on the downstream side, as this method has found favour with the engineers.

Table No.2

Methods of heightening Gravity Dams

- Direct Heightening
- Strengthened heightening
  - Strengthening by Conventional Methods
    - Upstream Strengthening
      - Use of seperate R.C.C.slab on the u/s-
      - Bonded slab of concrete or masonry.
    - Downstream strengthening
      - Bonded slab of concrete or masonry.
        - Direct Bonding
        - Deferred Bonding.
      - Sliding backing slab of masonry or concrete.
      - Earth Backing.
    - Complete earth envelopment with the dam forming the core-wall.
  - Strengthening by special methods
    - Strengthening by prestressed cables
    - Strengthening by use of hydraulic jacks

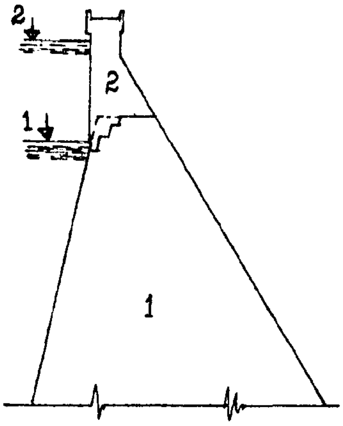



FIG 2.1.1
DIRECT HEIGHTENING

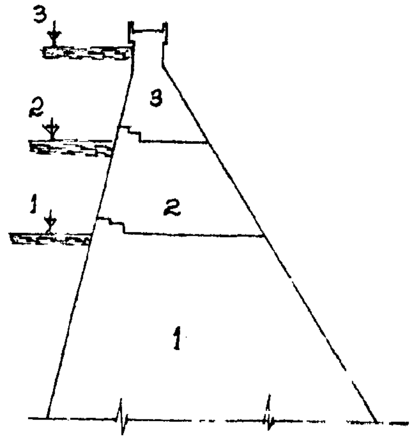



FIG.2.1.2
DIRECT HEIGHTENING

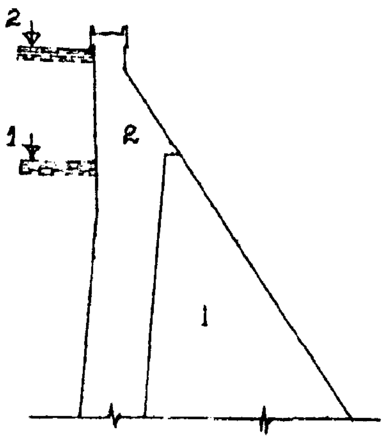



FIG.2.1.3
UPSTREAM THICKENING

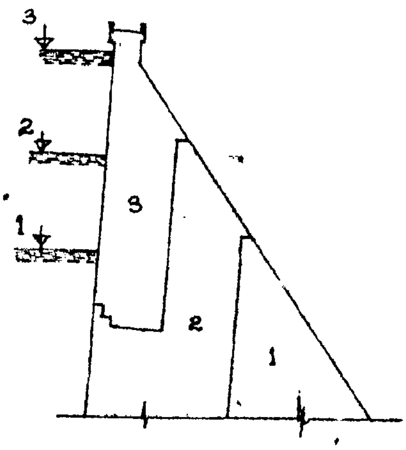



FIG 2.1.4
UPSTREAM THICKENING

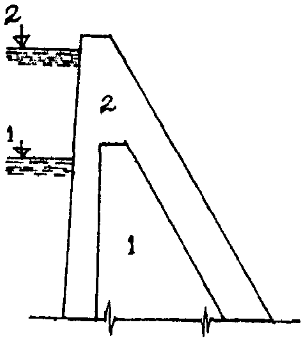



FIG.2.1.5
ENVELOPING

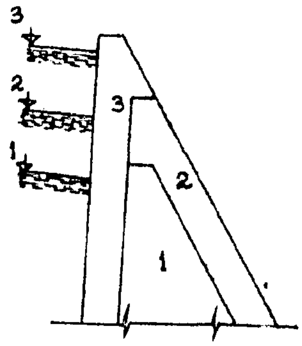



FIG.2.1.6
ENVELOPING

FIG 2.1
SCHEMES FOR HEIGHTENING OF GRAVITY DAMS.

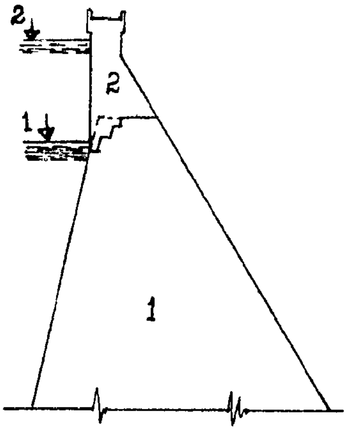



FIG 2·1·1
DIRECT HEIGHTENING

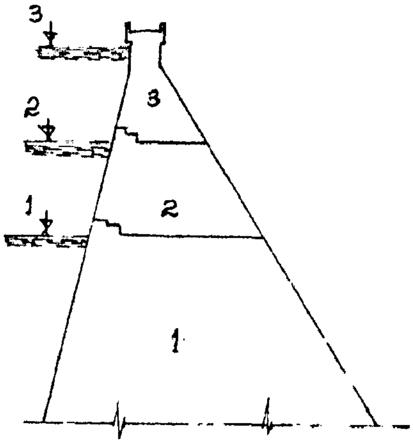



FIG 2·1 2
DIRECT HEIGHTENING

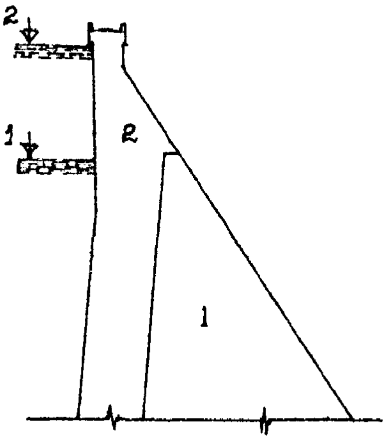



FIG .2·1 3
UPSTREAM THICKENING

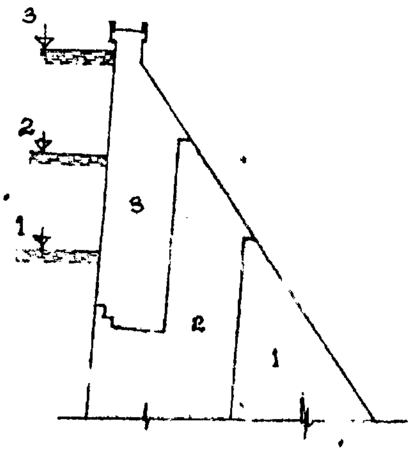



FIG .2·1 4
UPSTREAM THICKENING.

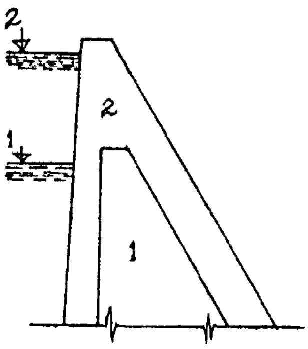



FIG .2·1·5
ENVELOPING

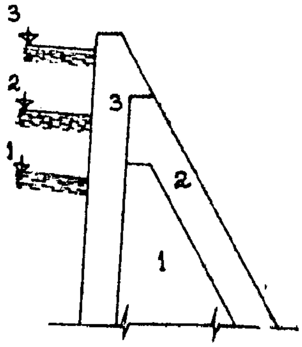



FIG .2·1 6
ENVELOPING

FIG 2·1
SCHEMES FOR HEIGHTENING OF GRAVITY DAMS.

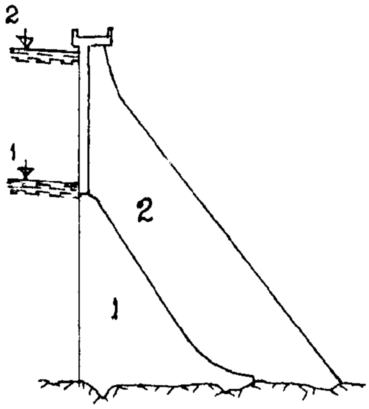


FIG. 2.2.1
D/S STRENGTHENING.

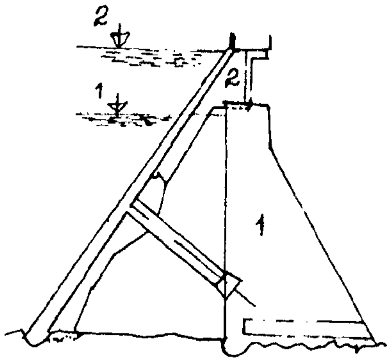


FIG. 2.2.2
U/S STRENGTHENING BY USING R.C.C. SLAB.

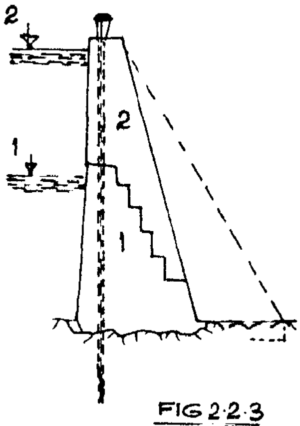


FIG 2.2.3
PRESTRESSED CABLING.

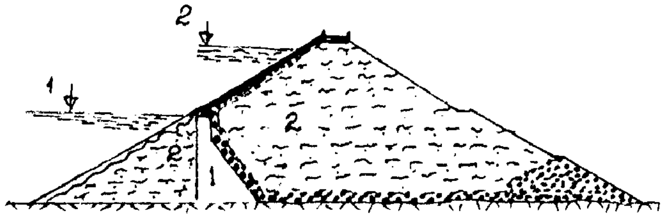


FIG 2.2.4
EARTH-ENVELOPEMENT.

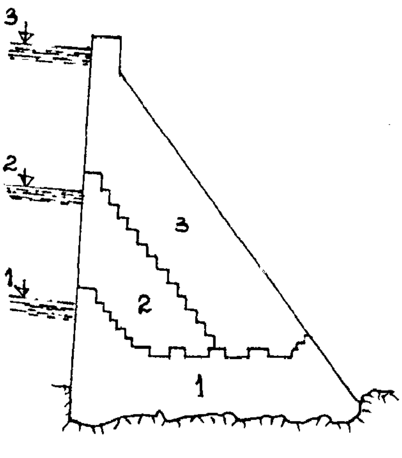


FIG. 2.2.5
CONSTRUCTION BY STAGES.

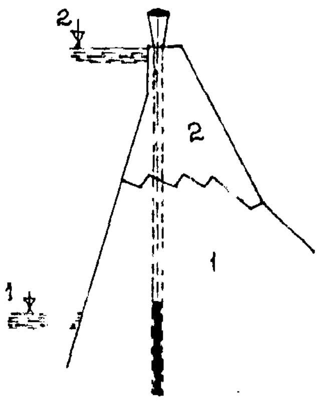


FIG 2.2.6
PRESTRESSED CABLING.

FIG 2.2
SCHEMES FOR HEIGHTENING OF GRAIVTY-DAMS

# CHAPTER III

# DESIGN CONSIDERATIONS IN HEIGHTENING AND STRENGTHENING OF DAMS

# CHAPTER III

## Design considerations in heightening & strengthening of Dams

### 3.1 Collection of Data

Having decided upon the necessity of strengthening a gravity dam for whatever reason, it is necessary to make investigations before proceeding with the design for strengthening and heightening. This is because one of the main considerations in raising or strengthening a structure is safety which can not be compromised with economic considerations. From this point of view the following types of investigations are generally made.

#### 3.1.1 Geological Examination

Geological examination of the reservoir area, above the existing full reservoir rim, which is further likely to be submerged on account of heightening of the dam is necessary. Geological examination of the foundation area that will support the strengthening section to be added will also be necessary to check its competency to withstand to the changed loading conditions. Knowledge of the geology of the reservoir area as well as that of the foundations of the existing dam and also information gathered during excavations for the foundations of the existing structure supplemented with the data collected from extra explorations undertaken on account of the proposed strengthening is of great value.

When the dam is to be heightened, the amount of raising may have to be limited in some cases from considerations of strength of the available rock foundations due to the presence of defects or weaknesses which are likely to persist in the rock masses inspite of special treatment to improve their properties. This aspect needs to be given proper attention. 

Additional head to be imposed on the foundations ~~xxx~~ or abutments due to raising may give rise to the need for grouting, drainage and other such safety measures. Hence this aspect may have to be studied in details. A detailed knowledge of the pattern of the joint system in the abutment and the foundation rock will also be essential so that the problems that may come up during excavation of rock under the influence of loading from the existing structure and reservoir head can be ascertained and necessary safeguards taken.

Detailed investigations regarding the various properties of the foundation rock such as specific gravity, porosity, compressive strength, modulus of elasticity etc. are especially important when the type and physical properties of the abutment rock under the new section to be added or of the foundation rock under the strengthening section to be added differ from the type of rock below the foundation of the existing structure. With this knowledge on hand, it becomes possible for the designer to make adequate safety provisions as well as provide for proper interaction between the old and the new parts of the structure by suitable design (8).

3.1.2 <u>Data on existing structure</u>:

Knowledge about the condition of the existing structure and also that of the materials of which it is made of is very important. Investigations may be conducted to get information on the following aspects.

(a) <u>Modulus of elasticity</u> of the actual material in the body of the dam. This can be determined from the measurements carried out in the laboratory on cored samples obtained from the body of the dam. It can also be determined in the field by measurement of velocity of propagation of shock waves artificially created, as they travel through the body of the dam.

(b) Samples of concrete collected during core boring can be used for determining the properties of in-situ concrete such as density, strength, impermeability etc. The bore holes can also be made use of in determining the quality of the joint between the foundation and the bottom-most concrete layers in the dam as well as for determining in-situ permeability of the concrete in the dam. They can also serve a useful purpose in determining the condition of horizontal construction joints and the extent and locations of leakages within the body of the dam.

(c) It is desirable to collect data regarding the deflections of the dam during operation as well as temperatures and stresses in the body of the dam and its foundations, if it is available from instruments embedded in the dam or can be conveniently obtained.

(d) At the time of designing the original dam section a certain uplift pressure distribution must have been assumed. However if the existing structure is old enough, uplift pressure observations collected over the elapsed life span of this structure may be available. Such data will provide a valuable guide for study and for reasonable and realistic estimation of uplift pressures which may be allowed for in the design of the structure for obtaining an adequately safe and economical structure after strengthening.

(e) It is necessary to inspect the drainage systems installed within the dam, spillway and other structures to check as to whether they are effective in their operation or not and whether they will be capable of relieving the additional hydrostatic pressures which will be created due to raising or whether some additional measures will be necessary.

(f) The records maintained regarding the observations made during periodical inspections of the dam can be of value in understanding its performance history and hence its condition.

The information so collected can serve as a useful guide in the design studies for the proposal of strengthening the dam.

3.1.3 Collection of temperature data

Following atmospheric temperature data needs to be collected:

1) Annual mean temperature @ the site,

2) Maximum and minimum temperatures,

& 3) Variation of temperatures during different seasons of the year.

This data is important as the cyclical changes of the surrounding atmosphere affect the placement temperatures of concrete as well as affect the temperature gradients in the cooling masses of placed concrete. The information thus collected will give an idea about the type and extent of the cooling arrangements which will be required to be provided during the construction stage whenever concrete backing is used.

Another important temperature observation is that of reservoir water. It is preferable to gauge the reservoir water temperatures @ various depths @ different locations so as to determine the levels at which water remains sufficiently cool all the year round so as to be of use in the construction stage for various purposes such as mixing water for concrete, circulation through embedded pipe system for effecting postcooling of concrete etc.

3.1.4 After completion of the investigations generally needed as stated above, the design criteria for strengthening of gravity structures can be decided. In the following paragraphs, the method of thickening by addition of concrete on the downstream side has been discussed.

3.2 <u>Thickening of Dam by addition of concrete or masonry on the downstream side</u>.

3.2.1 In case of heightening of gravity dams if a change only in the magnitude of loads acting is involved, all other conditions remaining the same, the new structure will be of nearly the same type as the existing one. The term 'all other conditions' includes the following :-

i) Topographic and geological conditions for the new section as compared to those for the original section,

& ii) Criteria of design.

(Condition number (i) is likely to be violated in those cases in which the height of the existing section had been limited by considerations of such local conditions).

The loading considerations for the dam constructed in parts as in the case of heightening are not the same as those in case the entire structure were built in one stage - i.e. monolithic in cross section for the entire height. In some cases, it may be possible to achieve the objective of enhanced storage by an upward extention of the water bearing surface. by raising the existing dam and by provision of downstream buttresses for support. This, however, may be applicable. for only comparatively low heightenings. For appreciable raisings, it shall generally be necessary to extend original gravity shape downstream to meet the safety criteria.

3.2.2 Construction for heightening can be looked upon to consist of two structural elements, namely

(i) the structural element needed to enhance the stability of the original dam under increased water pressure to which it will be later on subjected to. It is thus the structural element provided to strengthen the existing dam section to withstand the forces to be imposed after heightening in excess of those which were not anticipated in the original design, It is termed as the 'reinforcing element';

and (ii) the structural element above the top of the existing dam to retain the extra depth of water proposed to be stored against the structure to enhance storage capacity.

The latter element can be regarded as a conventional dam founded on the top of an existing dam instead of on the ground. Its design can therefore be dealt with by conventional methods.

The reinforcing element poses some problems in both design and execution and is therefore proposed to be discussed in some detail.

3.2.3 In the case of gravity dams, heightening may have to be done under two possible circumstances:

a) Foreseen Heightening,

& b) Unforseen heightening.

When heightening can be forseen at the time of construction of a dam at a particular site, it is generally economical to construct it in stages and to include some desirable features for raising of the dam in future in its initial planning, design and construction itself. This will permit confidence that, the final structure can be constructe for better structural behaviour than is possible if <u>such</u> provisions were not made. In case of unforseen heightening, the operations to be carried out will be more expensive as these will require a lot of extra care in view of danger to the original structure involved in executing them as well as difficult nature of their implementation.

It is proposed to confine the scope of the study mainly to the aspects of unforseen heightening or strengthening.

3.2.4 In the case of heightening of a dam the primary objective is to design it in such a way that, the raised dam will function structurally as if it had been originally constructed in one piece to its final dimensions. The best structural action will be obtained if this objective ~~dimensions.~~ is achieved. Such a design shall give ultimate economical section by appreciable reduction in cubic contents of the material required to be laid as compared to other alternative solutions However, obtaining a monolithic action will require special treatment measures, the cost of which must be considered in deciding upon an economical proposition.

3.3 <u>Problems involved in Thickening</u> :

3.3.1 The main basic difficulties involved in strengthening the section for obtaining a satisfactory action of the entire structure are as follows :

a) Provision of bond between the old and the new construction materials to obtain monolithic action is difficult.

b) It is difficult to keep the physical properties of the new materials the same as these of the old construction material and avoid influence of their difference on the stability of the dam.

c) The water in the reservoir may be standing against the dam during strengthening operations. This influences stability of the structure.

d) Foundation widening may create problems.

3.3.2 Great difficulty arises in securing an intimate and a strong bond between the added mass and the existing mass. This difficulty is more due to the difference in the age of the materials which is further made complicated in case of concrete due to its pecularities of thermal shrinkage, plastic behaviour and temperature strains. Another factor involved as regards elastic behaviour is the difference in the modulli of elasticity of the old and the newly laid materials of construction which affects load-sharing between the two.. Creep is yet another factor to be worthy of considerations, its important characteristic being its time-dependency.

3.3.3 Thus it is evident that, time is an important factor involved due to the fact that, while the material of the existing structure has attained stable conditions and therefore its final steady-state properties, the condition and properties of the ~~newly~~ newly laid material are in a state of continuous change with time. The above design aspects will now be discussed in the following paragraphs:-

3.4 <u>Provision of adequately strong bond at the junction of old and the new Sections</u>:

The basic assumptions in the gravity analysis of a dam section can be stated to be the following :-

i) Two dimensional isotropy of the material in the dam,

ii) Geometrical homotetics,

& iii) The internal secondary stresses from causes other than gravity and external forces are negligible.

These assumptions are not satisfied in either the strengthening or the heightening operations.

The first condition is not satisfied in such operations due to the variance between the elastic properties of the old and new construction materials and not too perfect a bond between the old and the new work. The third condition is not fulfilled due to imposition of shrinkage and temperature stresses on the section during setting of the new construction material, especially concrete. It is therefore necessary that, proper solutions to take care of the departure from the assumptions should be found to let the stress-distribution correspond generally with that estimated on the basis that, the design assumptions were truly satisfied.

In a monolithic structure, shear stresses can be transferred from one portion of its section to another. The conventional method of gravity analysis can be adopted in analysing such a structure provided the requirements noted earlier are also reasonably satisfied. However, if the section is jointed, shear stresses cannot be transferred completly across the joint and hence in such a case gravity analysis will not hold.

As concrete is generally adequately strong in compression and the design is generally so made that tensile stresses will not develop, the shear stresses may thus become critical in gravity analysis. Thus when the effect of additional forces to be resisted on the state of stresses is to be considered, shear stresses require special attention. A strong joint between the old and the new sections capable of resisting the shear stresse likely to develop in its region under the action of the additional forces is therefore necessary. In case it is not possible to get perfection in joining the two parts, as close an approximation to this condition as is possible to achieve must be sought, so that the joints or cracks occuring are still capable, either through whatever cohesion may remain or through friction, of transferring the deformation of one element to the other to obtain resisting action as one structure.

3.4.1 The bond-strength of the joint between the two sections can be considered to consist of two parts -

i) Cohesion component,

& ii) Friction component.

When cohesion component of this bond is likely to be poor, resistance due to friction assumes considerable importance in resisting the shear stresses. It is thus important that, the effective shear stresses at the joint as given by the formula,

$$T_{eff} = T - N. \tan \emptyset,$$

are kept lower than the cohesion at the joint between the two concretes. Here 'Teff' denotes the effective or residual shear stress, 'T' denotes the total shear stress on the junction plane, 'N' the normal stress on the junction plane and 'Ø' the angle of internal friction at the junction.

It would be evident from following discussion that, if the joint happens to be favourably located, the above requirement is easy of attainment.

3.4.2 On any plane passing through a point in a stressed mass two types of stresses act. One is the normal stress, which can be ~~ei~~ either compressive or tensile and the other is the shearing stress. Considering the governing loading condition, we can draw the type of stress picture at any such point, as shown in figure 3.1. Herein the curve in full lines represents the compressive stresses that can occur on the planes passing through the point under consideration, while the dotted curve represents the shear stresses that can occur on different planes passing through the same point.

As the frictional force always acts in a direction normal to that of the normal force on a plane and is equal to the product of the normal force and the tangent of the angle of internal friction of concrete, we can therefore draw the friction reaction diagram (R) as shown in figure 3.2. In this figure, the shear stress diagram (T) has also been superimposed. The shaded area represents the zones in which the shear stresses exceed the frictional resistances and thus the remaining zones in which the effective shear stresses do not exist are automatically demarcated & thus can be determined at any point in a stressed mass (10).

If the joint plane is passing through these zones of negative effective shear stresses at every point on it, monolithic action between the two sections will be ensured inspite of the joint and shear stresses will be effectively transferred between the old and the new sections.

3.4.3 In case of forseen heightenings, the downstream face of the Ist stage section can be located within the zone having zero effective shear. However a surer proposal would always be to locate the downstream face along the trajectory of either the major principle stress or the minor principle stress corresponding to the reservoir full condition in the ultimate stage. In case of Burguillo dam in Spain for example, the joint was kept along trajectory of the major principle stress while in case of Puentes-Viejas dam in the same country, the joint has been located for part length along trajectory of the minor principle stress. Please see figure 3.4.

In case of location of the downstream face along the trajectory of the major principle stress, the volume of work will be considerably more as the cross section so obtained will be appreciably bigger than the one needed for stability of the height of the dam as required for the Ist stage. To reduce the volume of work in the Ist stage and thus increase the benefits obtained by deferrement of larger part of capital expenditure to IInd stage, a better solution would be to determine the downstream slope as required for the Ist stage from stability considerations and make it stepped as shown in figure No.3.3, the surfaces of steps following the trajectories of the major principle stress. This will therefore ensure the desirable condition of zero shear at all points on the joint under reservoir full condition in the final stage. It will therefore be observed that by suitable location of the joint initially, monolithism between the two seperately constructed parts of the structure can generally be ensured.

3.4.4 However, strictly speaking, following the trajectory of the principle stresses while shaping the downstream face of the temporary section will not obtain the condition of entirely zero shear as stated above at the joint under reservoir full condition in the final stage in view of the factors given below .

i) Variation in lake levels at the time of bonding,

ii) Unforseeable and variable nature of hydrostatic uplift,

iii) Thermal condition of new concrete.

These factors are difficult to control. Their effect on the stress distribution is difficult to determine. This gives rise to uncertainty about actual conditions in the structure and therefore the method of shaping the downstream face of the Ist stage section along trajectories of the principle stresses can also not be regarded as the perfect method of truly achieving the condition of zero shear at the joint.

We therefore have to be content with the situation that, whether heightening is forseen or it is unforeseen, some shear stresses are always to be expected at the joint of the old and the new sections. The objective can however be attained better (by locating the joint plane in the zone of minimum shear) in case of forseen heightenings than in case of unforeseen heightenings when it can only be achieved as a matter of coincidence or chance.

3.4.5 From the discussion following, it will be seen that, parallel heightening even in unforseen cases of raising, if done to the recommended extent results in elimination of effective shear stress at all points on the joint and thus helps in ensuring monolithic action of the entire structure.

3.4.6 <u>Criteria for heightening from considerations of shear stresses due to external loads</u> :

In the case of unforeseen heightenings, it is evident that, even after ignoring the effects of the three factors mentioned in the earlier discussion, in most of the cases, the joint between the old and the new work can not attain the condition of zero shear stress, the downstream face of the existing dam not being along the trajectory of principle stress under full reservoir condition in the final stage. We shall therefore, determine the conditions under which either of the following states of shear on the joint can be satisfied for monolithic action :

i) Effective shear stress in all directions at every point of the joint is zero.

ii) Effective shear stress along the direction of the joint at every point of the joint is zero.

It is entirely the choice of the designer to adopt either of the two criteria mentioned above.

3.4.6.1 We can say that, for the first condition to be realised, the maximum effective shear stress at every point of the joint must be zero. In general, a maximum value is reached at each point accordir to lines which cut the trajectories of principle stress of the first order at an angle ( $\pi/4-\phi/2$), where $\phi$ is the angle of internal friction. For the purpose of illustration, the value of 'tan $\phi$' has been assumed as 0.75 in the following discussion.

In the case of a triangular profile with vertical upstream face and 'm' as the slope of its downstream face (please refer fig.35) it can be shown that, the condition for the maximum effective shear stress (Teff) at any point (x,y) to be zero can be stated as follows:

$$(N_1 - N_2)^2 + 4T^2 = (N_1 + N_2)^2 \sin^2\phi \quad \ldots\ldots\ldots (1)$$

Here, $N_1$ denotes the horizontal normal stress,

$N_2$ denotes the vertical normal stress,

& T denotes the shear stress,

at a point (x,y) in the body of the dam (9).

The expressions for $N_1$, $N_2$ & T are - -

$$N_1 = y$$

$$N_2 = \left(-\frac{w}{m} + \frac{2}{m^3}\right)x + \left(w - \frac{1}{m^2}\right)y$$

$$\& \ T = \frac{x}{m^2}$$

Here 'w' denotes the density of the material of the dam in tonnes/$m^3$.

$\therefore$ Putting these values of $N_1$, $N_2$ & T in equation (1), we get the condition in the following form :

$$-A\left(\frac{x}{y}\right)^2 + 2B\left(\frac{x}{y}\right) + C = 0$$

Where

$$A = \frac{+2.559}{m^6} - \frac{1.875}{m^4} + \frac{3.385}{m^2}$$

$$2B = \frac{-2.556}{m^5} + \frac{3.38}{m^3} - \frac{0.5}{m}$$

$$\& \ C = \frac{0.639}{m^4} - \frac{0.22}{m^2} - 2.245$$

Here $\tan \phi = 0.75$ & $w = 2.3$ t/$m^3$ have been assumed.

Thus for any given value of the downstream slope 'm', the equation can be solved. The equation shows that, the boundary $T_{effective} = 0$ is a straight line passing through the vertex of the dam profile. Its slope depends upon the value of 'm'. A plot of 'm' versus 'x/y' gives a curve as shown in figure 3.5. From this plot it can be seen that, in case of a triangular dam profile, for downstream slopes flatter than 0.695:1, there are two zones within the dam section, zone I, in which effective shear stress is negative and zone II in which the effective shear stress is positive. Please refer figure No;3.6 in this connection.

It can also be seen from the plot that, for values of downstream slope in the range 0.645 to 0.695, there exist three zones within the dam section. In the central zone, effective shear stress is negative. The central zone is flanked by the other two zones in which the effective shear stress is greater than zero. Please refer to figure No.3.7 in this connection.

For downstream slopes steeper than 0.645:1, the effective shear stress is positive @ every point within the entire dam section. However as the downstream slopes of dams are generally always flatter than 0.66:1, this case is not of any significance.

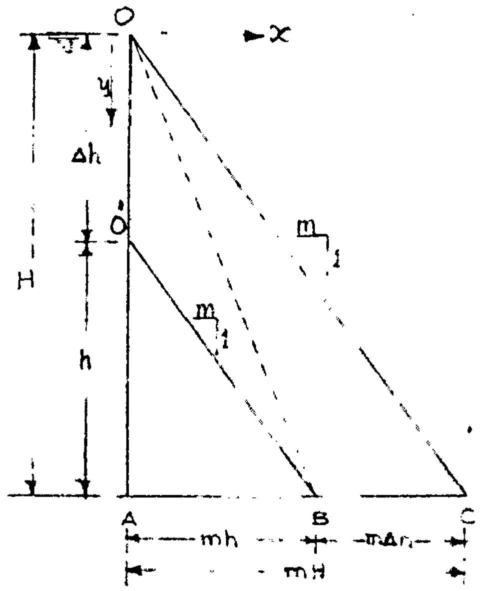
From the plot, it can also be seen that, the relationship between 'x/y' and 'm' is practically straight line for values of 'm' greater than 0.70:1. This relationship can be stated as follows:-

$$\frac{x}{y} = m - 0.393$$

or

$$\frac{x}{y} = m - 0.40 \text{ approximately.}$$

From this relationship, it is possible to determine the minimum enlargements which are necessary to satisfy the first criterion (10). This can be done as follows:-




In the above figure, O'AB is the existing dam section. OAC represents the section after heightening. Correspondigg to the reservoir full condition for the enlarged section OAC, OB represents the upstream-most limit for the location of the boundary for Teffective equal to zero in order that the first criterion is satisfied @ every point on the joint O'B. Thus when the boundary Teffective = 0 occupies the location OB, we get the minimum heightening needed to satisfy the first criterion.

As seen above, the equation for OB then will be,

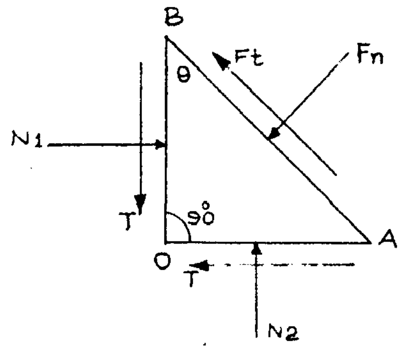
$$\frac{x}{y} = m-0.40 \quad \text{i.e.} \quad \frac{mh}{H} = m-0.40$$

$$\text{or.} \quad \frac{H}{h} = \frac{m}{m-0.40} \quad \text{or,} \quad \Delta h = H-h = \frac{0.4}{(m-0.4)}h$$

This relationship gives the minimum height by which the dam must be raised for satisfying the first criterion if adopted in the design.

3.4.6.2 If we adopt the second criterion for design in case of parallel heightening, we can analyse as follows :-

At any point in a dam, the normal and tangential stresses on planes parallel to the downstream face of the dam can be obtained as follows. Please refer to the figure given below :-




The stresses, normal and tangential, acting on the planes OA & OB at the point 'O' are known. The stresses Fn & $F_t$ acting on the plane BA parallel to the downstream face of the dam, are to be found out.

Resolving the forces in a direction normal to the plane BA, we get,

$$F_n \cdot BA = N_1 \; BO \cdot \cos\theta + N_2 \cdot OA \cdot \sin\theta - T \cdot OB \cdot \sin\theta - T \cdot OA \cdot \cos\theta$$

TAN θ = m = SLOPE OF DOWNSTREAM FACE OF THE DAM.

ASSUMING OB = 1, WE GET, OA = m & $BA = \sqrt{1 - m^2}$

$$\therefore \cos\theta = \frac{1}{\sqrt{1+m^2}} \quad \& \quad \sin\theta = \frac{m}{\sqrt{1+m^2}}$$

HENCE THE EQUATION GIVEN ABOVE CAN BE WRITTEN AS,

$$F_n \cdot \left(\sqrt{1+m^2}\right) = N_1 \cdot \frac{1}{\sqrt{1+m^2}} + N_2 \cdot m \cdot \frac{m}{\sqrt{1+m^2}} - T \cdot \frac{m}{\sqrt{1+m^2}} - T \cdot m \cdot \frac{1}{\sqrt{1+m^2}}$$

THIS REDUCES TO,

$$F_n = \frac{N_1}{1+m^2} - \frac{N_2 \, m^2}{1+m^2} - \frac{2 m \cdot T}{1+m^2}$$

Substituting values of $N_1$, $N_2$ & T in this equation we get,

$$F_n = \frac{y}{1+m^2} + \frac{m^2}{1+m^2}\left\{\left(\frac{2}{m^3} - \frac{w}{m}\right) x + \left(w - \frac{1}{m^2}\right) y\right\} - \frac{2m}{1+m^2} \cdot \frac{x}{m^2}$$

Putting $w = 2.3$, we get,

$$F_n = \frac{y}{1+m^2} + x\left\{\frac{2}{(1+m^2)m} - \frac{2.3m}{1+m^2} - \frac{2}{m(1+m^2)}\right\} + y\left\{\frac{2.3m^2}{1+m^2} - \frac{1}{1+m^2}\right\}$$

$$= \frac{y}{1+m^2} + \frac{2.3m^2.y}{1+m^2} - \frac{y}{1+m^2} - \frac{2.3mx}{1+m^2}$$

$$\therefore F_n = \frac{2.3m\,(my-x)}{1+m^2}$$

Similarly, it can be shown that,

$$F_t = \text{Tangential stress on plane BA}$$

$$= \frac{1}{1+m^2}\left\{1.3x - \frac{x}{m^2} - 1.3my + \frac{y}{m}\right\}$$

If (x,y) is a point on the construction joint of heightening we have,

$$my - x = m \cdot \Delta h$$

Substituting this value of 'my-x' in the expressions for $F_n$ & $F_t$, we get,

$$F_n = \frac{2.3\,m^2 \cdot \Delta h}{1+m^2}$$

$$\& \quad F_t = \frac{\Delta h}{1+m^2}(1/m - 1.3m)$$

The effective shear stress @ the point under consideration will then be given by ,

$$T_{eff} = F_t - F_n \times \tan \phi$$

$$= F_t - 0.75\, F_n.$$

$$\therefore T_{eff} = \frac{\Delta h}{(1+m^2)}\left\{1/m - 1.3m - 1.728m^2\right\}$$

For Teffective along the joint to be zero @ every point of the joint, we must have,

$$\frac{\Delta h}{1+m^2} \left\{ 1/m - 1.3m - 1.728\, m^2 \right\} = 0$$

or $\quad 1.728\, m^2 + 1.3\, m - 1/m = 0.$

Solving we get,

$m = 0.64.$

For values of $m > 0.64$, Teff works out negative.

Therefore we can say that, if the downstream slope of a dam of triangular profile is flatter than 0.64H:1v, the effective shear stress @ every point on the joint in a direction parallel to the joint will always be less than zero irrespective of the extent of raising, when the downstream face of the new section is kept parallel to the downstream face of the existing section.

The above discussion can be summarized by referring to the figure no.3.8 as follows. If the dotted lines shown on this figure represent lines of equal maximum effective shearing stresses, namely, T1', T2' etc., the profile Po starting at the foot of the line Teff= 0 will fulfill the first criterion i.e. no effective ~~xxxxxx~~ tangential force acting at any point on the joint in any direction. In case of profiles P1, P2, P3 etc. these will have in their respective joints maximum effective shear stress values of Teff=T1', Teff= T2' etc. occuring. Length of the joint affected in each such case will be that between its junction with the foundation and the point of its intersection with the line OAO. However, at any point on the joint, the shear force on the plane of the joint will always be less than the frictional resistance in case of all the profiles.

3.4.6.3 In some cases, it may not be possible to satisfy the condition of restricting the effective shear stress to less than zero (i.e. negative). In such cases, proper joint-treatment at the junction plane is warranted. The effective shear stress has then to be resisted by a combination of the following measures:-

(i) Careful concreting at the junction plane to achieve good bond,

(ii) Provision of shear keys,

&(iii) Provision of dowel bars.

The strength of bond between the two concretes depends upon the type of concrete mixes used, the degree of roughness between the surfaces in contact, care taken in placement of new concrete against the old, age of the two concretes and any special provisions added to strengthen the resisting action at the joint in the form of shear keys and dowels.

It is desirable to conduct field experiments to determine the strength of the joint against shearing and pull out under the effects of the different provisions or treatments proposed to be given for deciding upon the necessity and the extent of such provisions.

Results of field tests conducted at Koyna dam site, Bhandardara dam site in Maharashtra State and results reported by Mr.C.M.Roberts (21) are enclosed as Appendices I, II & III at the end.

The aspects of provision of shear keys & dowels for resisting residual shear stresses on the junction plane have been covered under paragraph 5.8.8.1 of Chapter V.

3.4.6.4 In Spain and some other countries, it is a general practice to consider the possibility of buckling of the added mass also. To avoid this tendency, they follow the rule that, the slenderness ratio of 'L/e' for the added mass shall always be less than 9, where 'L' is the length measured along the inclined axis of the added mass and 'e' is its thickness measured perpendicular to its downstream face. In order that, the slenderness ratio should be less than 9, it is necessary that for a profile having a downstream slope of 0.75H:1v, the following relationship holds -

$$\Delta h \geqslant \frac{h}{4}$$

In other words, to avoid buckling, raising to the extent of at least 25% of the original dam's height is considered preferable in any case (10).

3.4.6.5 Thus to conclude this part of the discussion we can say that,

i) If we desire to adopt the criterion that, the effective shear stress @ every point on the joint between the old and the new concrete in all directions should be zero, then in case we adopt parallel heightening, the dam will be required to be raised to the extent of at least $\left(\frac{40}{m-0.4}\right)$% of the original height of the dam.

ii) If we desire to adopt the criterion that, effective shear stress @ every point on the joint between the old and the new concrete in a direction of the plane of the joint should be less than zero, parallel heightening is desirable as it fulfills this requirement without putting any condition of minimum or maximum heightening, provided the downstream slope of the section is greater than 0.64:1.

It will be seen that, for the usual downstream slope of 0.75:1, the first criterion requires raising to the extent of 1.14 times the original height of the dam, which is excessive indeed. The information on heightening of dams given in the Table No.1 indicates that, generally the raisings done are between the range of 10% & 50% of the original height of the dam. This criterion is thus neither practicable nor necessary as it is rather too rigid. The requirements of criterion requiring no effective shear stress along the plane of the joint can be regarded as quite adequate and is therefore recommended to be adopted.

3.4.7 Effect of shrinkage of newly laid concrete @ the joint:

Having considered development of stresses at the joint due to heightening, we shall now consider the shear stresses likely to develop @ the joint due to thermal shrinkage of the new concrete when placed directly on the downstream face of the existing dam.

It is necessary that cracking at the joint is avoided which is the main objective. Thermal shrinkage of the new concrete is therefore a very important factor in heightening or strengthening of dams and therefore it deserves careful consideration. To appreciate its significance under the conditions of placement of new concrete directly against the concrete of the old dam, it is worthwhile to review the mechanism of cracking in mass concrete in general and that in new concrete placed over the old concrete in particular.

3.4.7.1 New mass concrete starts rising in temperature due to heat of hydration soon after placement. During this process, it changes from a soft yielding material with no resistance to plastic deformation to a hard semi-elastic material. While the concrete at the boundaries of a monolith in contact with the atmosphere tends to cool and contract, the concrete in the interior is not affected by it and it tries to expand due to heat of hydration. The colder contracting outer concrete tries to resist the expansion of the inside concrete which results in crack formation in the cold concrete. With lapse of time the concrete in the interior of the mass hardens with loss of plasticity and also starts contracting due to cooling. The delayed contraction of the interior mass is resisted by the outer concrete resulting in extention of the already formed boundary cracks towards the interior (23).

Effects of constraint within the new concrete mass itself on account of differential cooling thus causes boundary cracks especially when the temperature drops are high. This may also impose stresses in the older mass unless special care is taken to cool the concrete initially and close temperature control of new concrete effected to keep down the temperature rise in the process of setting. This will also restrict the temperature gradient with respect to the atmospheric temperatures to minimum and thus avoid development of cracks due to temperature stresses.

3.4.7.2 Now if the new concrete is placed directly on the old concrete of the dam, then in the early stages of setting and temperature rise, restraint to the expansion of the new concrete at the joint will be offered by the cold old concrete of the existing structure, but because of the relatively plastic nature of the young concrete, this will cause development of only insignificant compressive stress in the new concrete. When the new concrete begins to cool, it would have sufficiently hardened. In the process of cooling, it will try to contract but will be prevented from doing so by the restraint offered by the old concrete at the joint. This will give rise to tensile stresses in the new mass. As the compressive stresses which occured in the initial stages were low, such tensile stresses will be significant and will cause intense shearing stresses in the new mass at the joint tending to break it .

3.4.7.3 Having considered this effect qualitatively, an approximate quantitative analysis of this phenomenon is discussed below.

The freshly laid concrete after its initial set is subjected to shrinkage, which can be considered to comprise of three types :-

i) Drying shrinkage,

ii) Autogeneous shrinkage,

& iii) Thermal shrinkage.

The strains due to these shrinkages if not prevented are of the order of 0.06% to 0.08% in the case of drying and autogeneous shrinkage effects considered together and 0.02% to 0.03% in case of thermal shrinkage for a drop in temperature of $50^{\circ}$F. Thus considered together, the total shrinkage may be of the order of 0.11%. If we take modulus of elasticity of concrete for the purposeof illustration as $1.40 \times 10^5$ kg/cm$^2$, this shrinkage if totally prevented will result in producing a tensile stress of $1.40 \times 10^5 \times 0.11/100 = 154$ kgs/cm$^2$, approximately.

However due to the phenomenon of creep in concrete, stresses are not of this magnitude but may still be only of the order of 14 kgs/cm$^2$ due to the effect of drying shrinkage alone as calculated by Mr.F.G. Thomas (26). As strains due to contraction due to cooling are about one fourth of the strains caused by drying shrinkage, adding $\frac{1}{4} \times 14 = 3.5$ kgs/cm$^2$ to the stress of 14 kgs/cm$^2$, we may expect a total tensile stress of 17.5 kgs/cm$^2$ in the new concrete, if its total contraction is prevented completely by restraint at its joint with the old concrete over which it has been directly placed. As the restraint is applied tangentially to the joint by the adherence between the two concretes, restraining force on the new backing slab is the same as the shear force at the joint.

Assuming complete prevention of shrinkage throughout the thickness of the newbacking concrete, the shear stress at the joint for every one metre thickness of the backing concrete laid will be of the order of,

$$100 \times 17.5 = 1750 \text{ kgs/cm}^2 \text{ approximately.}$$

However the restraint goes on reducing progressively as we proceed from the joint towards the outer - face of the backing concrete due to shear deformation occuring within the new and the set concretes. This causes reduction in the shear stress at the joint to a considerable extent. Still the shear stress even after allowing for such reduction is likely to be of a sufficiently high magnitude as to exceed the shear strength of the joint (21).

3.4.7.4 In connection with the raising of Mullardoch dam in Scotland, tests were conducted to determine the strength of bond between two concretes having cement content of 212 kgs/m$^3$, which showed that, the joint could resist tensile stresses upto 10.5kgs/cm$^2$ and shear stresses upto 21 kgs/cm$^2$. These strengths are very inadequate, for resisting the internal stresses set up due to prevention of shrinkage of concrete. Thus it will be seen from this analysis as well that the bond at the joint cannot be relied upon to resist shrinkage stresses. It may be mentioned that the shearing stresses are accompanied by diagonal tension which can cause cracking in directions other than the plane of the joint. This cracking originating at the joint can easily penetrate into the interior of the new mass (23).

In addition to the setting up of shearing stresses at the interface between the existing section and the added mass, the shrinkage of the new mass also creates tensile stresses on the upstream face of the dam. The magnitude of these stresses will depend upon the mass of the added concrete.

3.4.7.5. To summerize this aspect, it may be said that, when a comparatively thick mass of fresh concrete is placed on and allowed to adhere to a mass of older, cold concrete, the contraction due to drying out and cooling under restraint can create internal stresses of high magnitudes in the combined mass in the vicinity of the joint of the two concretes, which are in addition to the stresses caused by externally imposed loads. These internal stresses can be sufficiently high to break the bond between the two concrete masses and thus can destroy the monolithicity of the two masses.

In the event of existence of bond at the joint, the shrinkage of new concrete can cause tensile stresses on the upstream face if placed directly on the old section as stated above. It is therefore desirable that, the new concrete should not be directly put against the old concrete at the time of placement, but that a sufficient time gap be allowed between the time of its placement and the time of its bonding with the old concrete, so that during this intervening period, the new concrete attains stable temperature conditions.

3.4.8 Other factors influencing stresses at the joint :

When the new concrete is placed on the old concrete directly, not only the prevention of shrinkage of concrete is involved creating internal stresses of concern but the prevention of the following movements is also involved which further enhances the severity of the undesirable conditions at the joint.

i) Elastic settlement due to the weight of concrete above the point under consideration , and

ii) Progressive creep of concrete.

Thus under the effects of elastic settlement as well as progressive creep, the new concrete mass has a tendency to move down along its joint with the old section. The stresses which may be caused by their prevention can be estimated by calculation of the amounts of movements so prevented. These movements can be considered either together or seperately. When considered together, their effect on stresses can be taken into account by appropriately reducing the value of modulus of elasticity of concrete to the so-called Effective elastic modulus. It is generally taken at one fifth of the original value.

It is, however, better to estimate these movements seperately owing to the fact that, the elastic deformations occur immediately as soon as the stress producing them is imposed, while the creep deformations proceed with increasing time towards an ultimate limiting value. Thus creep deformations are time dependent with the characteristic that the rate of increase of strain caused by creep decreases with time.

Glanville has given the following table to show the effect of the proportioning of a concrete mix on its elastic modulus (23).

Table No.3

| Mix proportion by weight. | Modulus of Elasticity in kgs/cm2 | Effective modulus of Elasticity (kgs/cm2) | |
|---|---|---|---|
| | | At the end of 12 months | Limiting value. |
| 1:1:2 | $3.72 \times 10^5$ | $1.19 \times 10^5$ | $0.985 \times 10^5$ |
| 1:2:4 | $2.39 \times 10^5$ | $0.535 \times 10^5$ | $0.444 \times 10^5$ |
| 1:3:6 | $1.405 \times 10^5$ | $0.33 \times 10^5$ | $0.282 \times 10^5$ |

As said earlier, it is better to estimate the true elastic strain seperately from the strain due to creep. As an illustration, in the case of a concrete mix having the proportion (1:2:4),

$$\text{as strain} = \frac{\text{Stress}}{\text{Elastic modulus}}$$

$$\therefore \text{Elastic strain per unit stress (kg/cm}^2\text{)} = \frac{1}{2.39 \times 10^5}$$

$$= 0.417 \times 10^{-5}$$

As effective modulus at the end of one year is $0.535 \times 10^5$ kgs/cm$^2$ (refer table No.3.)

$$\therefore \text{Combined strain at one year per unit stress} = \frac{1}{0.535 \times 10^5}$$

$$= 1.87 \times 10^{-5}$$

$$\text{Also combined strain in the limit per unit stress} = \frac{1}{0.444 \times 10^5}$$

$$= 2.25 \times 10^{-5}$$

$\therefore$ Strain due to creep alone $= (1.87-0.417)\times10^{-5}$

$= 1.453\times10^{-5}$ at one year.

& $(2.25-0.417)\times10^{-5}$

or $1.833 \times 10^{-5}$ in the limit.

In other words, if the backing concrete is directly placed on the old dam, downward movements due to elastic settlements as well as creep under the section of self-weight of the backing concrete shall be prevented introducing strains of the order of $1.833\times10^{-5}$ per $1 kg/cm^2$ of stress due to the phenomenon of creep alone. This will give rise to shearing stresses on the plane of the joint.

For instance, if we consider a point on the joint above which there is suppose 30 metre height of concrete, then,

$$\text{average unit stress} = \frac{30}{2}\text{metres} \times 2.40\ T/m^3$$

$$= 36\ T/m^2$$

$$\text{or } 3.6\ kgs/cm^2$$

$\therefore$ Elastic settlement $= (0.417 \times 10^{-5} \times 3.6) \times (30\times100)$

$= 0.045$ cms.

Limiting settlement due to creep $=$ Strain per unit stress x stress x length

$= (1.833 \times 10^{-5}) \times (3.60)\times(30\times100)$

$= 0.1977$ cms.

$\therefore$ Total settlement prevented

$= 0.1977 + 0.045 = 0.2427$ cms.

If complete prevention of settlement throughout the thickness of the concrete backing is assumed,

$$\text{Shear stress on contact plane} = \frac{0.2427}{30 \times 100} \times (2.39 \times 10^5)$$

$$= 19.35 \text{ kgs/cm}^2 \text{ per centimetre thickness of backing.}$$

Although full restraint to settlement may not act throughout the thickness of the added section, the above calculations indicate that shear stresses of large magnitude will develop on the contact joint in course of time, if bonding of the old and the new sections is done disregarding the effects of elastic settlement & the time dependent creep settlement. It needs to be noted that, the settlement due to creep is about 4.5 times the elastic settlement & hence it is necessary to consider the effect of the former while considering the important design aspect of ensuring the provision of adequate bond at the contact joint.

U.S.B.R. has given an approximate relationship between the creep and time in the form of a curve in 'Concrete Manual'. About 80% of limiting creep occurs within the first year, while most of the remainder takes place in the second year. It also depends upon factors such as age of the concrete when loaded, degree of saturation of concrete, grading of the aggregates etc.

Thus it will be seen that, the factors above discussed can also cause high internal stresses at the joint if new concrete is placed directly against the old concrete. The implications involved are thus obvious from the above discussions.

3.4.9 From this we can conclude that, placing of new concrete against the old section is highly undesirable due to the development of internal stresses of high magnitude at the joint caused by the restraint to shrinkage and settlement of the new concrete offered by the bond at the joint. However, the strength of bond at the joint being inadequate to resist these stresses, this will result in cracking and therefore a non-monolithic structural action. It is therefore necessary to defer the bonding of the two masses for a sufficiently long time so as to allow the above mentioned movements to occur. However, from the financial implications involved, an upper limit for such a time gap is required to be imposed. From the latter consideration, it may happen that some movements may still be remaining, in which case some stresses will have to be resisted by the bond at the joint. If the bond is not strong enough, other suitable provisions will be necessary.

3.5 <u>Stability under cracked condition</u> :-

3.5.1 It is thus difficult to obtain a strong bond at the joint due to the difference ~~in~~ in the behaviours of the two concretes principally due to their different temperature coefficients. In the event of a crack developing at the joint, the frictional resistance at the joint must be capable of fully absorbing the shearing stresses occuring in its region to ensure joint action of the two sections as one structure. However this reliance on friction as the resisting force for stability will be a dangerous proposition, if on creation of a crack due to break of bond, water finds its way into the cracked joint. This water can have two fold effects -

i) Hydrostatic uplift on the added mass,

& ii) Considerable reduction in the frictional resistance at the joint.

Even if the effect of hydrostatic uplift is disregarded, we find that, the stablizing moment gets reduced to be worthy of consideration (10).

3.5.2 Please refer to figure 3.9 (b). Considering a monolithic section and assuming that, the resultant of the forces of self weight of the triangular dam profile, uplift pressure and horizontal water pressure passes through the downstream middle third, we get,

$$\text{STABILIZING MOMENT ABOUT TOE} = \left(\frac{\gamma_c\, m^2 h^3}{3} - \frac{\mu\cdot m^2 h^3 \gamma}{3}\right)$$

$$= \frac{(\gamma_c - \mu\cdot\gamma)\, m^2 h^3}{3}$$

$$\text{OVERTURNING MOMENT ABOUT TOE} = \frac{(\gamma_c - \mu\cdot\gamma)\, m^2 h^3}{3\times \ldots}$$

Here 'h' denotes the final dam height and '$\mu$' the coefficient of uplift.

$$\therefore \text{FACTOR OF SAFETY AGAINST OVERTURNING ABOUT DOWNSTREAM TOE} = \frac{\text{STABILIZING MOMENT}}{\text{OVERTURNING MOMENT}}$$

$$= 2.$$

In case a crack is assumed to develop at the joint, the added section will still have a tendency to overturn about its downstream toe, while the old section will try to overturn about the old downstream toe.

Such a difference in the overturning actions causes a reduction in the stabilizing moment which is equal to the product of the reduction ....

~~ction~~ in the lever arm due to change in the axis of rotation and the weight of the section of the original dam.

Thus, if $\Delta h$ is the extent of raising,

reduction in stabilizing moment

$$= \left\{ \frac{1}{2} m\, H \times H\, (\gamma_c - \mu \cdot \gamma) \right\} m \cdot \Delta h$$

$$= \frac{(\gamma_c - \mu \cdot \gamma) m^2 H^2 \Delta h}{2},$$ where 'H' denotes original height of the dam.

$\therefore$ Stabilizing moment under fissured condition.

$$= (\gamma_c - \mu \cdot \gamma) m^2 \frac{h^3}{3} - \frac{(\gamma_c - \mu \cdot \gamma) m^2 H^2 \Delta h}{2}$$

$$= (\gamma_c - \mu \cdot \gamma) m^2 \left\{ \frac{h^3}{3} - \frac{H^2 (h-H)}{2} \right\}$$

$\therefore$ <u>Stabilizing moment after crack formation</u>
Original stabilizing moment for monolithic section

$$= \left\{ 1 - \frac{3(h-H) H^2}{2h^3} \right\}$$

$=$ 'R' say.

If we plot 'R' versus 'H/h', as shown in figure 3.9 (a) we see that 'R' reduces as H/h is decreased from 1 towards zero but after reaching the least value of 0.778 for $H/h = 2/3$, it again increases with decrease in H/h. When $H/h = 1$, $R = 1$ and when $H/h = 0$, $R = 1.00$.

Thus when $\Delta h = 0.5H$, or raising is 50% of the original height of the dam, maximum reduction in the stabilizing moment to the extent of 22.2% occurs and the factor of safety against overturning reduces from 2 for monolithic condition to 1.56 for the cracked condition.

This reduction in factor of safety is thus significant and worthy of notice. Although effect of friction has not been considered in the above analysis which will provide for better stability, we also can not forget that, we have not taken into account the opposite effect produced by the hydrostatic uplift at the joint. A well lubricated fissure at the joint will also cause tensile stresses at the base of the added section thus worsening the conditions further.

3.5.3 Thus it is evident that, a fissure at the joint will not by its existence alone be dangerous in reducing safety of the structure, but that its accompanying effects such as appreciable reduction in friction at the joint and hydrostatic uplift are likely to enhance the degree of severity to such an extent as to bring the structre on the brink of failure.

As a safety precaution to minimise the bad effects of entry of water into the joint in the event of crack formation, it is preferable to provide a properly planned drainage system on the downstream face of the existing structure in order to drain away and thus provide relief to the water which may seep under pressure into the cracked joint. These measures can be in the form of :-

i) Locating the contraction joints in the new concrete exactly at places where these are provided in the existing structure.

ii) A drainage gallery can be provided at a level sufficiently low in the added section.

iii) Placement of a system of semi-circular pipes on the downstream face of the existing dam. These can be connected to the new gallery.

iv) If there is appreciable seepage through the body of the dam finding way into the joint, it can be controlled and adequately reduced by proper grouting.

3.6 <u>Effect of differences in properties on the stability of the dam</u> :

3.6.1 Difference in the ages of the concrete of the old dam and the concrete of the section to be added is the cause of differnees in the elastic behaviours of the two masses. The effect on stress distribution within the sections can be determined analytically by the use of finite element method of analysis which takes into account the differences in the modulii of elasticity of the different parts of the structure. On the other hand, photoelastic studies can also be employed for this purpose.

3.6.2 Lower the value of modulus of elasticity of the new concrete, the larger will be the sharing of increased loads by the old section of the structure. This may cause tensile stresses on the upstream face of the dam. From these photo-elastic experiments, that value of elastic modulus of the new concrete can be determined for which, if the water level in the lake is raised from the maximum level at the time of bonding to the final reservoir level after heightening, the will be no tensile stresses on the upstream face of the dam. This value will depend upon the dimensions of the original section and the extent of rise in the water level (20).

3.6.3 Such a study was conducted in the case of raising Odomari dam in Japan. This dam had an original height of 62.5 m and it was raised by ~~XXX~~ 10 metres. Photo-elastic studies showed that, when the modulus of elasticity of new concrete reached a value at 80% of the elastic modulus of the existing concrete, tensile stress induced on the upstream face was of the order of 0.5 kgs/cm$^2$. only. It was also found that, the new concrete required three months to attain this value (20).

3.7 Influence of Reservoir level on stability

3.7.1 The most preferable condition for the strengthening of a dam will be when the reservoir is empty. However this is not an economical proposition as it results in loss of valuable hydro-electric power or irrigation benefits etc. depending upon the purpose of storage provided. Therefore it would be preferable if the normal reservoir operation could be maintained even during the construction of the strengthening section. This attempt to keep the dam in service during the construction of additional section poses some problems.

3.7.2 Thus, the special characteristic of loading in the case of heightened dams is that, the strengthening section is added upon the existing one which has already been strained and deformed under the effects of external loads. These are variable external loads, as the lake level changes during construction. Other factors which affect stress distribution and which also are variable with time are shrinkage, difference in age between the two concretes, etc. which have been considered earlier. All these factors which vary with time and affect stress distribution in the structure are difficult to control. If bonding is done simultaneously with heightening, the stress pattern created in the structure, which is difficult of accurate assessment on account of variable nature of these factors, will get locked up in the structure giving rise to uncertainty as regards the actual conditions in the structure. This method is therefore not preferred on this account also.

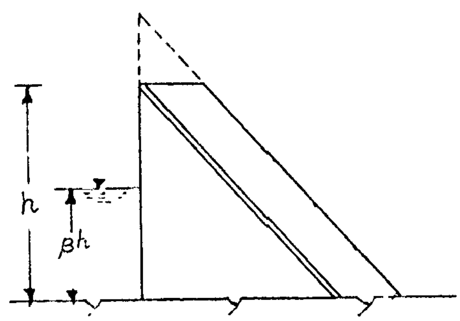
3.7.3 Thus when the addition is done to the existing section, let us suppose under lake full condition, the stress-distribution within the mass of the existing structure remains nearly unchanged. This is as the existing structure has fully deformed under the prevalent forces when the new mass was added. Hence there is practically very little sharing of loads between the two parts as far as the existing forces are concerned. When the water level rises to the new top, additional thrust acts which is resisted by development of stresses across the combined section which could result in tension on the upstream face, especially when the existing section itself does not satisfy the criterion of no tension under the action of forces which acted on it prior to its raising.

3.7.4 To avoid the development of tensile stresses on the upstream face, it is thus necessary to increase the load sharing between the two sections. This will be best achieved under reservoir empty conditions. However this is not permissible on account of the financial implications involved. The condition of making the reservoir empty will not however be needed to avoid tensile stresses on the upstream face.

3.7.5 The appropriate lake level at which the new section may be bonded with the old section can be theoretically calculated. This factor will certainly affect the framing of the construction schedule of the work. Thus it is necessary to keep the new work seperate from the old initially from this aspect also.

3.7.6 Before discussing how this permissible reservoir elevation at the time of bonding can be determined, we shall first discuss how xx stresses in the combined structure can be calculated.

3.7.6.1 We suppose that, the new structure is constructed seperate of the old right upto the top of the latter. The new work however derives its support for stability either through the anchor bars provided in both the works or through rib contacts with the old work. Let us suppose that there is water in the reservoir for a depth equal to '$\beta h$' and under this condition we propose to connect the old structure with the new one. Please refer the sketch given below.




Under these conditions, the existing dam is in a deformed but balanced state under the action of horizontal water thrust corresponding to the water depth '$\beta h$', uplift corresponding to the same water depth, weight of the old section itself and the component of the weight of the added section normal to the downstream face of the existing section. An additional force of friction acting downwards on the existing section will also come into play if the new mass derives its support by resting on the old section at few points through rib-contacts. Magnitude of this frictional force, which will be equal to the product of the normal component of the weight of the added section and the coefficient of friction at rib contacts, is an uncertain factor in such cases. So calculations are done in such cases by assuming possible maximum and minimum values of the coefficient of friction, so that the possible range in the variation of stresses is known.

The stresses at the base or any horizontal section of the dam can thus be found under the action of the above forces by conventional gravity analysis. Seperate analyses are carried out for the existing section and the added section. Now assuming that, the new mass has attained stable conditions from other considerations, if we bond the two sections under the above conditions of stresses existing in them, the stress distribution as existing at the time of bonding will be as if locked up within the structure. Therefore we term this stress picture as ''Locked in stresses''. The structure after sealing will behave as a monolithic structure as far as resisting of additional forces imposed upon it after this bonding are considered.

Thus there is superposition of the following stresses:-

i) The stresses which exist at the time of bonding the two sections corresponding to the then existing forces.

ii) The stresses produced in the whole structure behaving as one unit under the action of additional forces imposed upon it after bonding.

Thus it is possible to determine the stress picture under final conditions of loading. By analysing in this manner for different reservoir elevations at the time of bonding, that reservoir elevation for which the stress on the upstream face shall just be compressive can be determined.

3.7.6.2 In the case of simple profiles, it is possible to develop a general equation expressing the relationship between the maximum permissible reservoir elevation and the downstream slope of the dam for achieving the condition of no tension on the upstream face of the structure.

Mr.Masao Kondo, a Japanese Engineer connected with the work of heightening of 'Odomari Dam' in Japan gives the following formula(20) for deciding upon the downstream slope of the new section to be added for strengthening the existing section to satisfy the condition of no tension on the upstream face. (Please refer figure number 3.10 in this connection.).

$$A m_o^2 + B m_o + C = 0.$$

Where,

$$A = (1+s)^3 (\gamma_c - \mu\cdot\gamma) + (1+s)^2 \cdot \gamma \cdot a \ ,$$

$$B = (1+s)^3 (n\cdot\gamma_c + 2n\cdot\gamma - 2n\cdot\mu\cdot\gamma - \gamma_c\cdot k_1) + 2\gamma(1+s)^2 \cdot n\cdot a + (1+s)\left\{2(\gamma_s - 1)d^2\cdot n - 2\cdot\gamma\cdot n\cdot\beta^2\right\},$$

$$C = (1+s)^3\left\{\gamma\cdot n^2 - \gamma\cdot\mu\cdot n^2 - \gamma - 1.4\gamma\cdot k_1\cdot\xi^3 - \gamma\cdot n\cdot k_1\right\} + \gamma(1+s)^2\cdot n^2\cdot a + (1+s)\left\{2(\gamma_s-1)d^2\cdot n^2 - 2\gamma\cdot\beta^2 n^2\right\} + \gamma\cdot\beta^3(n^2+1) - (\gamma_s-1)\cdot d^3\cdot(n^2+C_e),$$

and

$$a = \frac{2n\cdot\beta^2}{m+n} - \frac{(1+n^2)\cdot\beta^3}{(m+n)^2}.$$

In the above formula, the notations are as under:

i) $m$ = Downstream slope of existing section supposed to be triangular.

ii) $n$ = Upstream slope of the existing section.

iii) $m_o$ = Downstream slope of the added section to be determined.

iv) $\gamma_c$ = Unit weight of concrete.

v) $\gamma$ = Unit weight of water.

vi) $\gamma_s$ = Unit weight of silt.

vii) $\mu$ = Coefficient of uplift.

viii) d = ratio of $\frac{\text{depth of silt}}{\text{Original height of the structure.}}$

ix) h = original height of the dam.

x) $\beta$ = $\frac{\text{Water depth during construction}}{\text{original height of dam.}}$

xi) s = $\frac{\text{Amount of raising}}{\text{Original dam height.}}$

xii) $\xi$ = $\frac{\text{Maximum water depth under final conditions}}{\text{Final height of dam.}}$

xiii) Ce = Coefficient of silt pressure.

xiv) kl = Seismic coefficient of horizontal earthquake acceleration.

This formula was used by the Japanese Engineers in case of raising of Odomari Dam in determining the downstream slope of the strengthening section for the condition of reservoir operation of lake full condition at all times. Thus by plotting 'mo versus $\beta$' curve, they determined the downstream slope for the value of $\beta = 1$ for no tension condition.

In view of the advantages the method of parallel heightening has in the absence of effective shear stresses at the joint, this author was interested in having the same downstream slope for the section to be added. From this point of view, the above equation was transformed by taking mo = m in an attempt to use it to calculate the maximum permissible reservoir elevation for no tension condition in case of parallel heightening. The use of this transformed equation was checked in an actual case assuming $\mu = 1$ as is always assumed as per recent design practice. The formula however did not give satisfactory results. As derivation of the above mentioned formula is not available, there is no way to further comment upon this formula.



Looking to the importance of this aspect however, a formula has been derived which is given below -

$$A\beta^3 + B\beta^2 + C = 0,$$

Where,

$$A = \frac{s(s+2)(n^2+1)}{(m+n)\ (1+s)^2}, \quad B = \frac{-2n \cdot s}{(1+s)},$$

$$\& \quad C = -\frac{\gamma_c}{\gamma}\left[\frac{s^2(2m+ms-n)}{(1+s)^2} + \frac{m^3(1+s-3s^2)+m(3s+1)-m^2ns(3s+2)}{(m^2+1)}\right]$$

$$- \frac{n(s+1)\ (2m+n)}{(m+n)} + \mu(m+n)(1+s) + \frac{(1+s)}{(m+n)}$$

$$- \frac{\gamma_s}{\gamma}\left[2nd^2 - \frac{d^3(n^2+ce)}{(m+n)}\right]$$

The notations in this formula are the same as before.

For a simpler profile of traingular shape with upstream face vertical, we get the following formula by putting n = 0,

$$\ldots m(1+s)^2 \Big/ s(s+2) \Big] \left\{ \frac{\gamma_c}{\gamma}\left[\frac{ms^2(s+2)}{(1+s)^2} + \frac{m^3(1+s-3s^2)+m(3s+1)}{(m^2+1)}\right] - \frac{(1+s)(\mu m^2+1)}{m} - \frac{\gamma_s}{\gamma}\,\frac{d^3 ce}{m} \right\}$$

Taking $\gamma_c = 150$ lbs/cft $\gamma = 62.4$ lbs/cft and $\mu$ = coefficient of uplift = 1, we can calculate different values of '$\beta$' for different heightenings for different downstream slopes. A plot of '$\beta$ versus s' for different downstream slopes is given in figure 3.12.

From this figure, the following observations can be made:.

a) In the case of each curve, there is a particular range of heightening in which the requirement of water level @ the time of bonding is very sensitive to the changes in the extent of raising involved. Beyond this range, however, the changes are smooth and gradual.

b) For any particular downstream slope, as the amount of raising is increased, the maximum permissible lake level at the time of bonding also increases. In other words, larger the amount of heightening greater will be the flexibility available in the construction programme.

c) As the downstream slope of the profile is flattened, lake level can be kept @ a higher level considering the same amount of heightening. This can however be expected without actual calculations also.

In the above calculations, a free and frictionless sliding joint has been assumed.

3.7.6.3 <u>Effect of friction on locked-in Stresses</u>

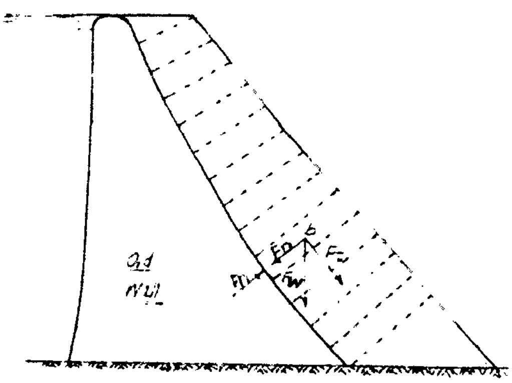
As mentioned earlier, when the new mass gets its support from the old section through rib-contacts, there may be certain amount of friction acting at this contact if the joint is not a perfectly sliding one. This frictional resistance at the joint may act as to prevent sliding down of the new mass over the old mass at these contacts. This restraint is difficult to determine. The effect is to cause a very marked discontinuity in stress at the toe of the old wall as shown in figure no.3.11 in the case of heightening of Koyna Dam in Maharashtra State which was at one time under consideration (35). The degree of this discontinuity in stress depends upon the frictional restraint at the joint, being the greatest when the friction is the least.

The effect of such a sudden change in the foundation pressures is difficult to predict, but obviously it will depend upon the magnitude of such discontinuity in pressures and the quality of the foundation rock

In the case of Mundaring wair in Australia, the stress analysis showed that, a friction-less joint caused a 50% increase in the compressive stress at the heel of the new wall as compared to the one which would have been caused had the structure been monolithic. Also minimum compressive stress at the heel of the old dam was developed when the friction along the joint was having a coefficient of 0.50 (23).

Sliding Joint Method of Analysis :

We saw earlier in an outline how stress analysis is carried out in the case of strengthening of dams. The method of such analysis is given below in more details as put forward and explained by Mr.V.C. Munt, an Australian Engineer (23). Please refer to the figure given below :



<u>Method</u>: The new wall is arbitararily divided into strips 10' wide and the centre of gravity of each strip is found graphically. It is desirable to take strips sloping as shown, because then the boundaries conicide more or less with the principal planes and therefore there is very little shear between each strip.

For the purpose of illustration, one strip will be considered in detail. The area and the weight of the strip are found. The point 'b' is the centre of gravity of the strip and the vector Fw represents its weight. Fw may be resolved into two components, Fn. & Ft, normal and tangential, respectively, to the slope of the old downstream face. This may be done for each of the strips comprising the new wall and the values of Fn and Ft scaled off and tabulated.

The old wall is considered to supply equal and opposite reactions to the normal components Fn at all points along the joint. Thus, we may distribute these normal forces along the length of the joint and find the normal stress between the new and the old concrete. In case of Mundaring weir it varied from zero at the top of the joint to about 1.96 kgs/cm$^2$ at the bottom. If there is a 1.22 m wide rib in the centre of a 15.25 m wide monolith, it should take about 5/8th of the total load and hence the normal pressure between the rib and the old wall is :

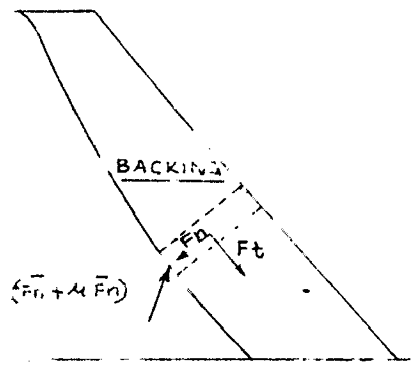
$$\frac{5}{8}\left\{\frac{1.96 \times 10 \times 15.25}{1.22}\right\} \text{ t/m}^2$$

$$= 153 \text{ t/m}^2 \text{ or } 15.3 \text{ kgs/cm}^2$$

In some places the ribs may shrink away from the old wall and so transmit no stress. Therefore, there will be other places which transmit more than their fair share of the load, so that pressures greater than 15.3 kgs/cm$^2$ are likely to occur.

Considering frictional forces along the joint, if the coefficient of friction is '$\mu$' then at each point where a normal force Fn is applied, there will be a corresponding frictional force $\mu$·Fn acting along the joint. The direction in which it acts depends upon whether we are considering the old or the new wall, being upwards on the new wall and downwards on the old wall.

The forces acting on each strip of the new wall are shown below :-

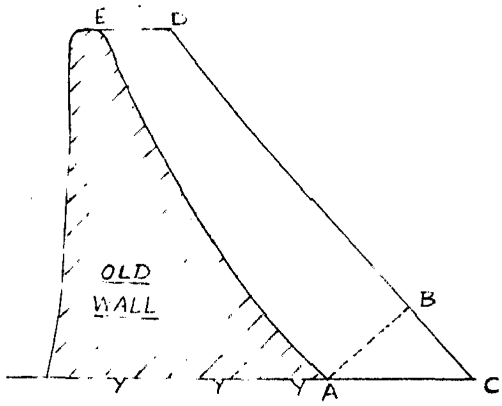



The stresses on the base of the new wall and the line of resultant pressure may be found in the usual way considering the external forces Fn and $\mu$·Fn and the weight Fw of each strip. This can be done to study the effect of different coefficients of friction.

The effect on the old wall of the new wall leaning against it may be determined by considering the newly imposed forces Fn and $\mu$·Fn.

It is not necessary to divide the new backing section into strips when the downstream slope of the dam is linear and not a curved one.

3.7.6.4 In this method of dividing the new section into strips, the effect of the weight in the portion ABC (Please refer figure given below) in producing a normal force on the downstream slope of existing section is not considered.




The whole mass of the new section, i.e. ACDE is acting as one single monolithic structure and hence the total normal force acting on the downstream face of the existing section shall be the component of the total weight of the section in the normal direction. However neglect of the portion ABC is on the safe side.

### 3.8 Foundation Widening

3.8.1 It has become possible to use big rounds in blasting near structures by the adoption of multiple row blasting with short delay ignition, by new methods for calculating the charge required to ensure loosening of rock without unnecessary throw and by increased experience concerning ground vibrations and how these can be reduced in large rounds. When blasting in the proximity of structures, the "ground vibrations" is often the factor which finally decides how the blasting is to be carried out.

As the ground vibrations set up by blasting operations for foundation widening in case of strengthening of dams are likely to set up unsafe vibration velocities in the dam, it is essential that their effects are considered in planning of the work. The risk of damage as a function of the size of the charge and of the distance between the charge and the structure to be protected has to be determined in such cases.

3.8.2 Mr.U. Langefors and Mr. B.Kihlstrom (48) from their experimental experience have given the relation $Q/R^{3/2} = \text{constant}$. This ratio is termed as the charge level. The value of the charge level permissible in a particular situation depends upon the type of rock which is to be excavated and the type and condition of the structure safety of which is under consideration. 'Q' represents the charge in one hole in kgs. or the sum of several simultaneously fired charges at the same distance 'R' in metres from the structure. The charges obtained corresponding to the applicable charge levels in a particular case on the basis of their recommendations are on a very conservative side. The permissible charge levels in case of excavation in granitic rock in proximity of normal houses founded on same rock for different degrees of risks have been given by the above authors as in the table given below. In this table 'C' is the velocity of propagation of waves in the foundation rock. The charge levels can be selected from this table depending upon the degree of safety desired. These charges are applicable beyond a distance of 2.5 times the depth of the charges from the structures to be protected. For lesser distances, the charge level will be $\frac{Q}{R^2} = \text{constant}$.

Table No.4

Relation between charge level and vibration velicity 'v' in hard rock.

| $Q/R^{3/2}$ kg $m^{-3/2}$ | V mm/s | v/c $\mu$/m | Description of damage in normal houses. |
|---|---|---|---|
| 0.008 | 30 | 6 | Fall of plaster. No cracking. |
| 0.015 | 50 | 10 | No evidence of cracking. |
| 0.030 | 70 | 14 | No noticeable cracks. |
| 0.060 | 100 | 20 | Insignificant cracking. (Threshold value) |
| 0.120 | 150 | 30 | Cracking. |
| 0.25 | 225 | 45 | Major cracks. |
| 0.50 | 300 | 60 | (Fall of stones in galleries and tunnels). |
| 1.00 | - | - | Cracks in rock. |

Note: In softer materials, a certain v/c value requires 2-4 times larger charges.

As the charge levels given are for normal houses, a higher charge level can be allowed for structures of better type of construction for the same degree of risk or damage. Thus for concrete structures, a value of charge level of 0.12-0.25 may be adopted instead of 0.06 for 'insignificant cracking'.

3.8.3 It has been shown from experiments that, the damage caused depends upon the relative vibration velocity. Hence either charge level or relative vibration velocity (i.e. v/c) can be used to define the different degrees of damages. Generally it is recommended to keep the vibration velocity below 70 mm/sec. (2.8 in/sec) in case of rock and 50 mm/sec. in ground with a lower propagation velocity.

This limitation in the charges that can be used in the proximity of structures necessitates the use of short delay detonators and intelligent use of scattering time involved in the ignition times of such delay detonators. The use of proper delay detonators is termed as arranged interference, while the use of scatter in increasing the charge to be fired at the same instant is termed as the unarranged interference. The adoption of these techniques do not require reduction in either the size of the individual charges in the holes or the total quantity of charge in a round. The total quantity of charge in a round may however be restricted on account of the number of delay detonators available in a series. Reduction of ground vibrations by these techniques will now be discussed below.

3.8.4 The ground vibrations caused by a single charge acquire a maximum amplitude only after one or several previous minor deflections. The vibration is generally of short duration and in most cases, only three full vibrations can be expected to have an amplitude greater than 50% of the maximum amplitude. All others can be ignored. Therefore, at intervals greater than 3T, where T is the time of one cycle of vibration, there is no collaboration between two different shots.

When the delay interval '$\tau$' is less than 3T, there is cooperation between the charges if $\tau$ is an integral multiple of T. Mathematically the condition for arranged interference can thus be stated to be :-

$$n\tau = K\,T,$$

Where 'K' must be an integer but not the ratio between K & n i.e. $K/n \neq 1,2,3$.

Here 'n' denotes the number of intervals in a round. In words, the above equation implies that, the spread of the round in time, which is'n$\tau$' shall be distributed over one or a number of full periods of the vibration concerned. In such a case, the different wave systems weaken one another resulting in reduced ground vibrations.

Thus with interval times, which are small in relation to the natural vibration time, the number of intervals is chosen large enough to ensure that 'n$\tau$' amounts to the time of at least one full period. If it does not become possible to adopt the interval times exactly so that 'K' becomes an integer, then it can be shown that, as large a part of the entire round as corresponds to the nearest integral value fulfills the interference condition and only the excess part cooperates.

3.8.5 Unarranged interference is obtained if the time spread for a number of charges is large enough to cover at least a full period of the vibrations, even if this period is distributed quite arbitrarily. A ground vibration will be obtained which is reduced to half or less, according as the spread increases to higher values than the vibration time. In order to calculate the effect of unarranged interference, the reduction factor 'p' at different frequency values, the natural vibration period (reciprocal of frequency) and the values of the scattering ( $\pm\Delta\tau$ ) are given in the following table. This table applies to a number of shots which are detonated at the same interval number with scattering of ' $\pm\Delta\tau$ ' which is so defined that 5/6 of all values of the scattering lie in an interval of $2\cdot\Delta\tau$ ms. For instance, in case of short delay detornators, the scattering for the two lowest may be $\pm$5 ms, for the three succeeding ones may be $\pm$10 ms, and still greater for higher intervals. For one second interval detonators, the scattering may be $\pm$100 ms or more.

Table No.5

| f c/s | T msec. | $\Delta\tau = \pm 6$ | $\pm 10$ | $p$ $\pm 25$ | $\pm 100$ | $\pm 200$ms |
|---|---|---|---|---|---|---|
| 5 | 200 | 1 | 1 | 1 | 1/2 | 1/3 |
| 10 | 100 | 1 | 1 | 1 | 1/3 | 1/6 |
| 20 | 50 | 1 | 1 | 1/2 | 1/6 | 1/6 |
| 50 | 20 | 1 | 1/2 | 1/3 | 1/6 | 1/12 |
| 100 | 10 | 1/2 | 1/3 | 1/6 | 1/12 | 1/25 |
| 200 | 5 | 1/3 | 1/6 | 1/6 | 1/25 | 1/50 |
| 500 | 2 | 1/6 | 1/6 | 1/12 | 1/50 | 1/100 |

Note: Reduction factor ( $p$ ) is for the total charge ( Qi) within one and the same interval in unarranged interference. The vibration effect is assumed not to be lower than the value which is obtained with a single hole.

Nowif the charge allowed as per Table 4 given in the beginning is Q, the charge quantity in the same interval Qi, the reduced charge by use of Table number 5.. is Qred, then it is necessary that,

$$Q_{red} \leq Q$$

where,

$$Q_{red} = p \cdot Q_i$$

$$\therefore \; p \cdot Q_i \leq Q$$

$$\text{or} \quad Q_i \leq Q/p$$

The above is a sufficient condition,

when $\tau > 3T$ also.

If $\tau < 3$ T, cooperation may take place in the different groups, each one of which is represented by Qred. This cooperation does not occur between the interval numbers which have a time distance of 3 T or more. If $\tau = T$, for example, the intervals 1,2 and 3 will co-operate, but not 1 and 4 or higher numbers nor will 2 and 5 etc. Therefore the condition in this case will be,

$$3 \cdot Q_{red} \leqslant Q.$$

If the interval times are less than T, then generally $\Delta\tau < \frac{T}{2}$ and therefore reduction factor $p = 1$. Then, the total charge in the interval Qi cooperates but a reduction can still be effected between the different intervals by a regularly arranged interference by the use of the condition,

$n\tau = K\,T$, where K is an integer but not K/n.

This will not allow the effect to exceed that from an individual interval.

For the particular problem in hand, it is desirable to determine the permissible charge level by actual experimentation in the field as its value to be adopted depends upon the type, condition and importance of the structure, the type of rock available at the site etc.

Thus blasting for excavation can be done in the close proximity of the dam without endangering the safety of the structure. By exercising proper care in the use of blasting charges and the use of proper blasting techniques, its cost can be kept down to the minimum possible.

3.9 <u>Mix design for concrete</u> :

As regards the concrete mix it is preferable to use the same sources of materials of construction for manufacture of concrete which were used in the construction of the existing structure.

The points which need special attention while designing the concrete mix for the new section are -

(a) Heat of hydration and unit water content should be kept to the minimum to minimise shrinkage effect.

(b) The concrete should have sufficient initial plastic property to allow effective pipe cooling without cracking.

(c) As the concrete becomes old, its strength should increase gradually.

These properties in concrete can be achieved by taking the following measures:-

i) Use of moderate heat cement for mass concrete.
ii) Use of good fly ash to replace cement.
iii) Reduction of the total amount of cement and fly ash to the maximum possible extent.
iv) Use of dispersing agent.
v) Strict control on aggregate manufacture to obtain concrete of uniform quality.

Concrete near the exterior may have higher heat development.

3.10 Before concluding this chapter, other problems associated with strengthening may now be mentioned which also require a careful consideration on the part of the designer.

3.10.1 Ingenuity is needed on the part of the designer to attain stability of the structure and its parts and also to ensure satisfactory operating conditions for the control works. For instance,

i) Increase in head could produce velocities in some parts of the outlet systems that may cause cavitation, vibrations, or otherwise undesirable flow conditions,

ii) Shape of a crest or intake designed for original head may be unsuited for increased head.

3.10.2 Control structures may be preferably checked by hydraulic model studies to ensure that changed flow conditions will not cause certain discharges to jump out of the stilling device or over training walls etc.

3.10.3 In use of equipments such as for instance large gates and their hoists, it may be very difficult to redesign them to accomodate a system of loads and forces different from the one for which they were originally designed.

3.10.4 In case of turbines and generators, their limited range of operating characteristics needs to be carefully considered and the design of critical parts of their components e.g. bearings need careful checking for their satisfactory function under new conditions.

3.11 To conclude this chapter, we arrive at the following conclusions from the above discussion.

i) Adequate geological and other investigations are needed before taking up design of strengthening and raising.

ii) It is preferable to place the new concrete seperate from the old section initially.

iii) When the new concrete section is supported on few ribs resting on the old section, it is preferable to have as frictionless a joint as is possible to obtain, when foundations are of g ood sound rock.

iv) The reservoir elevation at which the new section is bonded with the existing section has an important bearing on the final stress distribution in the structure. Therefore the construction programme needs to be so framed by taking into consideration variations in the reservoir levels during the year that, delays in the e xecution on account of non-realisation of the bonding level for a considerable period of the working seasons are avoided.

v) At the time of bonding the old and the new sections, the new mass should have attained stable temperature conditions and most of the settlements due to various factors should have occurred.

vi) It is necessary that, the bond at the joint is made as strong as possible and adequate enough to prevent cracking under the effects of residual stresses and external loads.

vii) The reservoir level may be allowed to rise above the bonding lake level preferably after the new concrete has attained the necessary rigidity for adequate load sharing between the old and the new section without development of undesirable stresses.

viii) Efficient and adequate temperature control is essential to reduce the time gap between the placement of new concrete and its subsequent bonding with the old section in order to reduce the period of construction.

ix) The design of concrete mix needs to be so done as to obtain the desirable properties in the concrete.

x) Proper drainage system should be provided on the downstream face of the existing section to eliminate the possibility of hydrostatic uplift.

xi) Hydraulic model studies need to be conducted to check up satisfactory operation of control structures which require changes during the heightening operations.

xii) In some cases, it may be preferable and economical to increase the height slightly more than the requirement in order to obtain a sufficiently high permissible reservoir level at the time of bonding the two sections. This is apparent from the curves given in figure 3.12. A higher permissible lake level at the time of bonding is desirable to easen the framing of the construction programme and to avoid financial losses in certain cases.

xiii) Utmost care is needed in planning the blasting operations for foundation widening to avoid any harm to the structure. This needs experimentation to decide upon the safe charge level to be used as the same depends upon the type and condition of the structure and the foundation rock at the site.

Appendix I

RESULTS OF EXPERIMENTS CONDUCTED TO DETERMINE BOND STRENGTH OF JOINT BETWEEN OLD AND NEW CONCRETE IN CONNECTION WITH "STRENGTHENING OF KOYNA DAM BY CONCRETE BACKING" (MAHARASHTRA)

| Sr.No. | Shear Stress in p.s.i. | Normal stress in p.s.i. | Cohesion in p.s.i. | Average Cohesion in p.s.i. |
|---|---|---|---|---|
| (A) BLOCKS CAST ON ROUGHENED SURFACE: | | | | |
| 1 | 267 | 63.0 | 177 | |
| 2 | 230 | 59.5 | 145 | |
| 3 | 265 | 68.2 | 168 | |
| 4 | 253 | 67.5 | 162 | |
| 5 | 208 | 39.0 | 152.5 | 155.0 |
| 6 | 267 | 66.6 | 172 | |
| 7 | 217 | 46.5 | 150.5 | |
| 8 | 232 | 50. | 160.5 | |
| 9 | 190 | 41 | 131.5 | |
| 10 | 190 | 37 | 137.5 | |
| 11 | 233 | 47.7 | 165 | |
| 12 | 183 | 35 | 133 | |
| 13 | 224 | 65.2 | 131 | |
| (B) BLOCKS CAST ON UNROUGHENED SURFACES- | | | | |
| 1 | 150 | 32.4 | 104 | 83.0 |
| 2 | 175 | 44.5 | 111.5 | |
| 3 | 77.3 | 17 | 53 | |
| 4 | 91 | 19.4 | 64.5 | |

Notes: i) The cohesion has been worked out taking the angle of internal friction as 55°

ii) The blocks were tested after curing for 28 days.

RESULTS OF SHEAR TESTS OF BHANDARDARA DAM SITE (MAHARASHTRA)

| r.No. | | Shear stress in $kg/cm^2$ | Normal stress in $kg/cm^2$ | Bond strength (cohesion) in $kg/cm^2$ for $\phi = 45^\circ$ | $\phi = 55^\circ$ |
|---|---|---|---|---|---|
| | (A) Concrete blocks cast on Semi-wet inclined surface | | | | |
| 1. | | 11.00 | 3.02 | 7.98 | 6.70 |
| 2. | | 4.20 | 0.85 | 3.35 | 2.98 |
| 3. | | 9.40 | 0.33 | 9.07 | 8.93 |
| 4. | | 4.50 | 1.28 | 3.22 | 2.68 |
| | (B) Concrete blocks cast on complete wet inclined surface. | | | | |
| 5. | | 4.12 | 1.17 | 2.95 | 2.45 |
| 6. | | 5.64 | 1.64 | 4.00 | 3.30 |
| 7. | | 4.50 | 1.28 | 3.22 | 2.68 |
| | (C) Concrete blocks cast on dry inclined surface | | | | |
| 8. | | 9.35 | 3.08 | 6.27 | 4.95 |
| 9. | | 7.38 | 2.06 | 5.32 | 4.44 |
| 10. | | 3.62 | 1.39 | 2.23 | 1.64 |
| 11. | | 5.95 | 1.15 | 4.80 | 4.32 |
| 12. | | 5.30 | 1.37 | 3.93 | 3.35 |
| 13. | | 8.80 | 2.92 | 5.88 | 4.54 |
| | | | Average = | 4.79 | 4.08 |

Notes: 1. Curing period = 28 days for concrete blocks.

2. Concrete for blocks-premixed(1:1½:3) with maximum size of aggregate ¾", water cement ratio =0.60 to 1.

3. '$\phi$' denotes the angle of internal friction at the contact joint.

RESULTS OF SHEAR TESTS OF BHANDARDARA DAM SITE (MAHARASHTRA)

| Sr.No. | | Shear stress in $kg/cm^2$ | Normal stress in $kg/cm^2$ | Bond Strength (cohesion) in $kg/cm^2$ for $\phi = 45^\circ$ | $\phi = 55^\circ$ |
|---|---|---|---|---|---|
| | (A) Masonry blocks cast on dry inclined surface. | | | | |
| 1. | | 2.98 | 0.86 | 2.13 | 1.77 |
| 2. | | 3.71 | 1.17 | 2.54 | 2.04 |
| 3. | | 6.07 | 1.58 | 4.49 | 3.83 |
| 4. | | 3.16 | 0.89 | 2.27 | 1.90 |
| 5. | | 3.55 | 1.03 | 2.52 | 2.08 |
| 6. | | 2.96 | 0.91 | 2.05 | 1.65 |
| | (B) Masonry blocks cast on complete wet inclined surface. | | | | |
| 7. | | 5.34 | 1.18 | 4.16 | 3.66 |
| 8. | | 3.42 | 0.73 | 2.69 | 2.38 |
| 9. | | 2.78 | 0.63 | 2.15 | 1.89 |
| 10. | (C) Masonry blocks cast on semi-wet inclined surface. | 3.76 | 1.02 | 2.74 | 2.32 |
| 11. | | 3.02 | 0.70 | 2.32 | 2.03 |
| 12. | | 5.73 | 1.26 | 4.47 | 3.93 |
| | | | Average = | 2.88 | 2.45 |

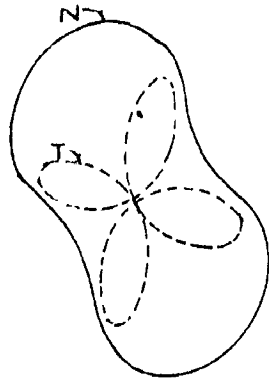
Notes: 1. Curing period = 28 days for masonry blocks.

2. Masonry in c.m. (1:3) proportion, water cement ratio = 0.60 to 1.

3. '$\phi$' denotes the angle of internal friction at the contact joint.

Adherence of prepacked concrete to set concrete.

| Sr.No. | Type of Test | Average shear strength of joint in lb/sq.in. |
|---|---|---|
| 1. | By pressing on a 6-inch cube of prepacked concrete formed between and supported by, two 6-inch cubes of concrete... | 271 |
| 2. | By pushing horizontally on a 20x10x10-inch block of prepacked concrete formed on a concrete surface... | 50 |
| 3. | By torsion applied to a 9-inch cube extracted from the finished dam with the concrete joint running in the middle ... | 217 |
| 4. | By torsion applied to a 12-inch diameter core with a 1.5-inch deep groove ground down at the concrete to prepacked concrete joint... | 259 |



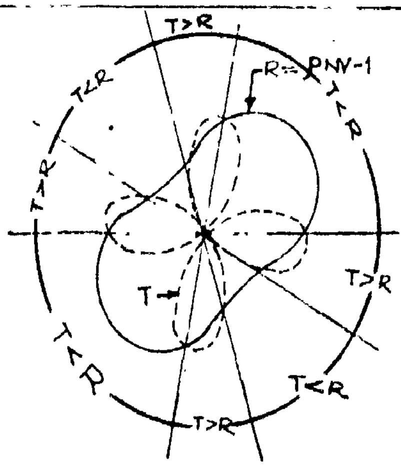




FIGURE-3·1

POLAR SCHEMATIC DIAGRAM.

FIGURE - 3·2 (a)

POLAR SCHEMATIC DIAGRAM.

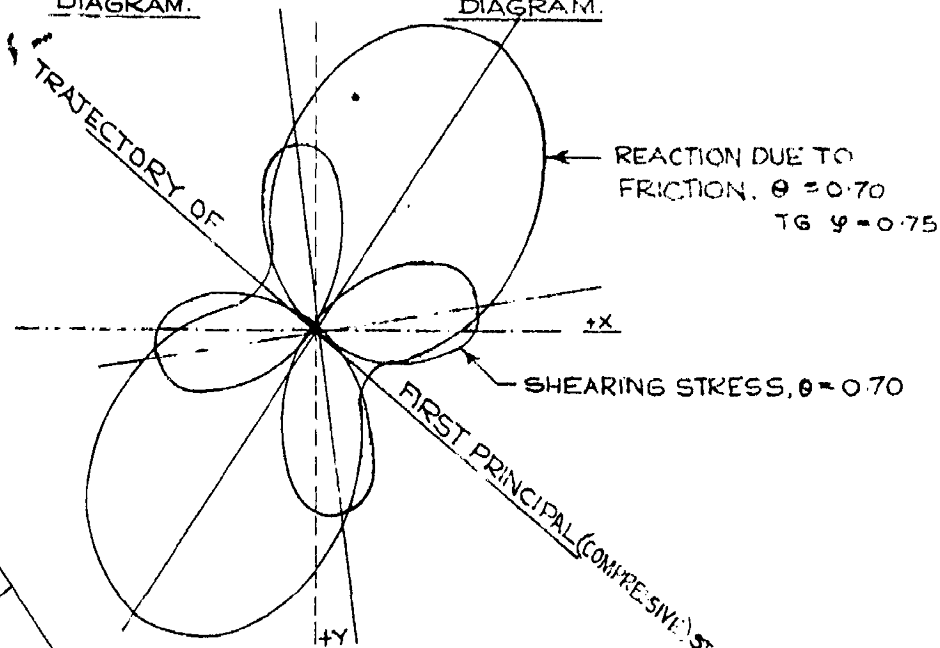
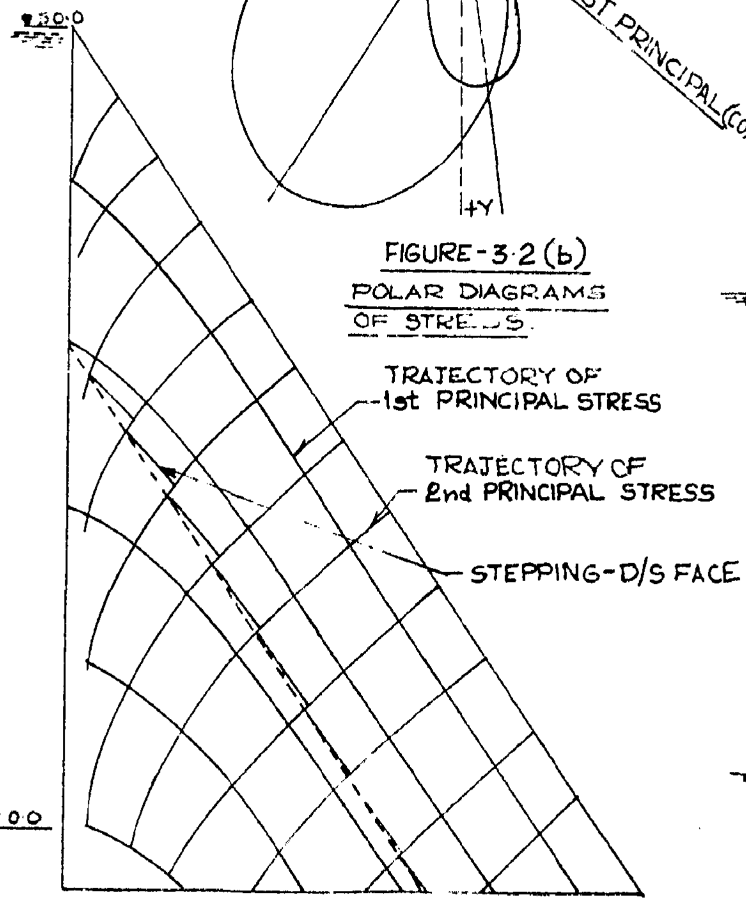



FIGURE - 3·2 (b)

POLAR DIAGRAMS OF STRESS.



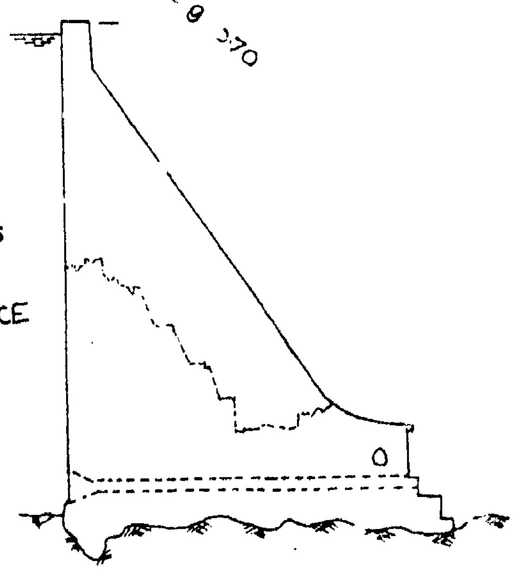




FIGURE - 3·4

TYPICAL SECTION OF BURGUILLO DAM CONSTRUCTED IN TWO STAGES

FIGURE - 3·3

PROVISIONAL PROFILE WITH DOWNSTREAM FACE STEPPED ALONG ISOSTATICS.

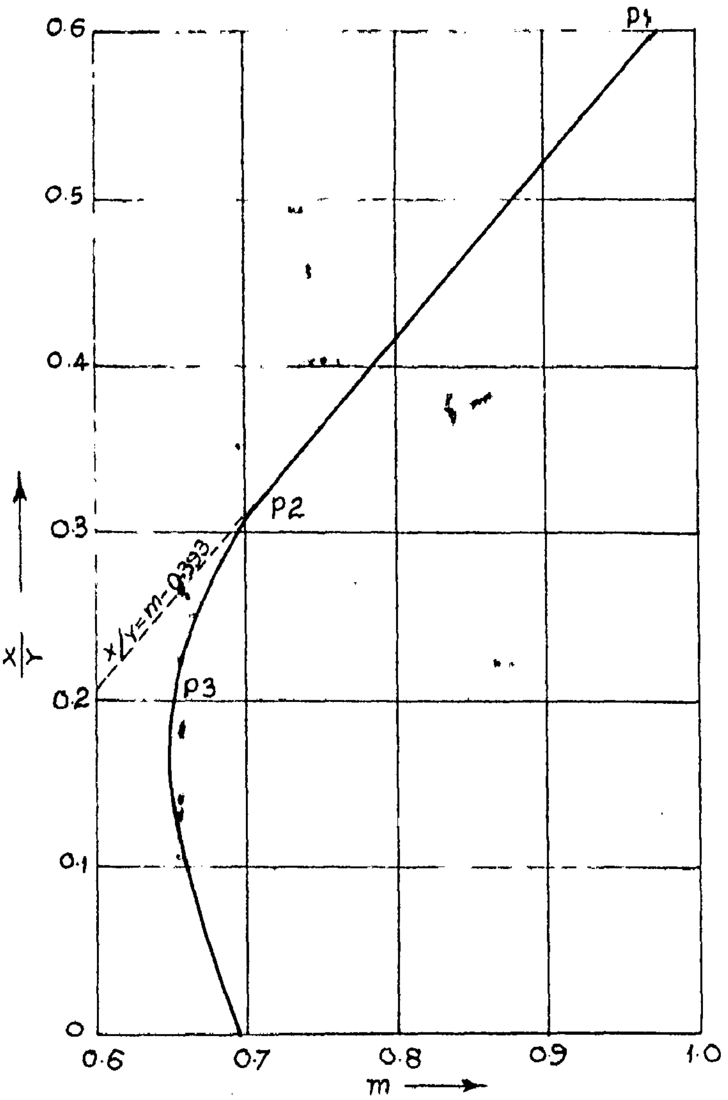



FIGURE 3·5

DIAGRAM DETERMINING BOUNDARY OF ZERO EFFECTIVE SHEAR.

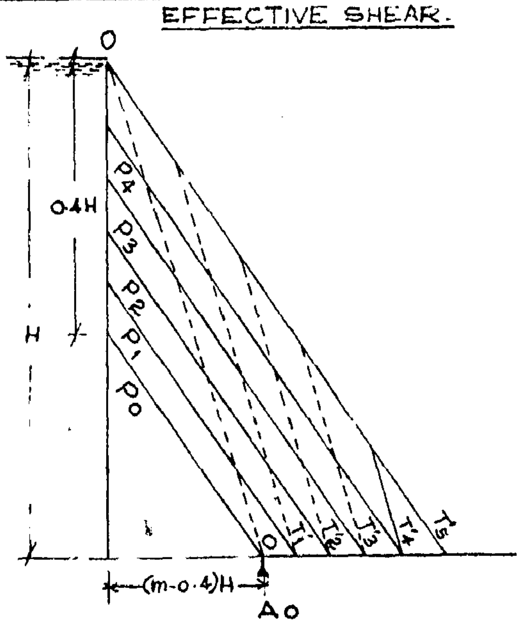
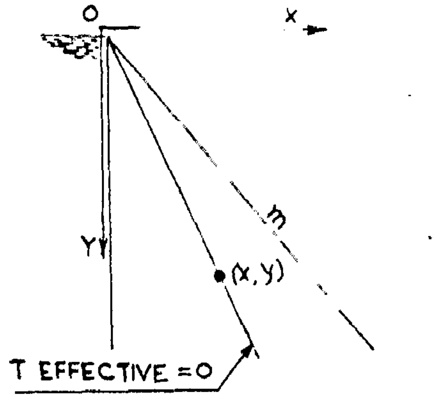



FIGURE 3·8

PROFILES $P_0$, P ...... ETC. AND LINES OF EQUAL EFFECTIVE SHEARING STRESS.




KEY - DIAGRAM FOR FIGURE · 3·5

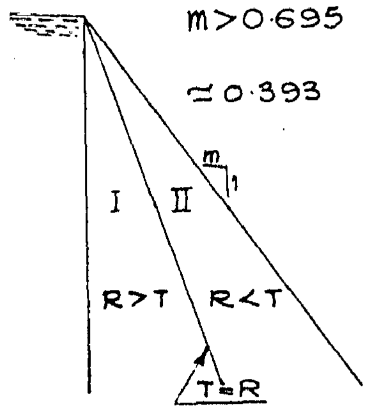



FIGURE 3·6

IN AREA I EFFECTIVE SHEARING STRESSES ARE NEGATIVE. CONTRARY IN AREA II.

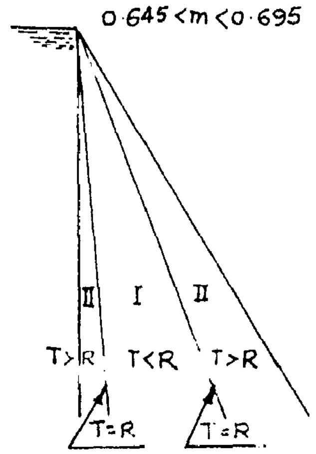



FIGURE 3·7

EXISTENCE OF TWO AREAS (II) IN WHICH EFFECTIVE SHEARING STRESS IS POSITIVE.

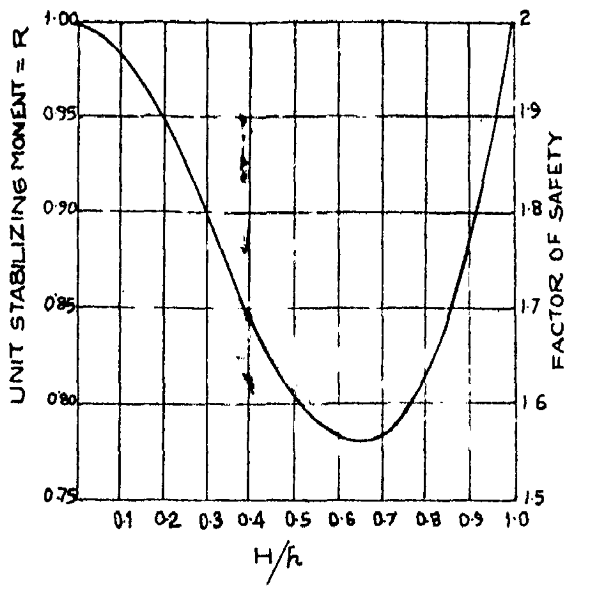



FIGURE 3·9 (a)

FACTORS OF SAFETY AGAINST OVERTURNING FOR DIFFERENT HEIGHTENINGS WHEN A CRACK OCCURS AT THE JOINT.

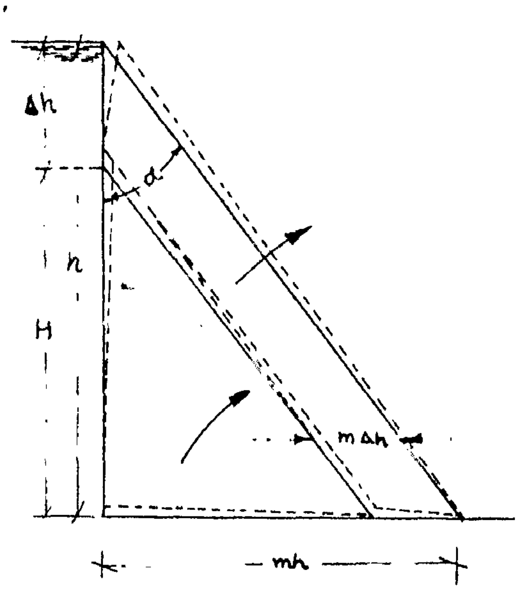



FIGURE - 3·9 (b)

FISSURES WITHOUT FRICTION AT THE JOINT DECREASE STABILIZING MOMENT.

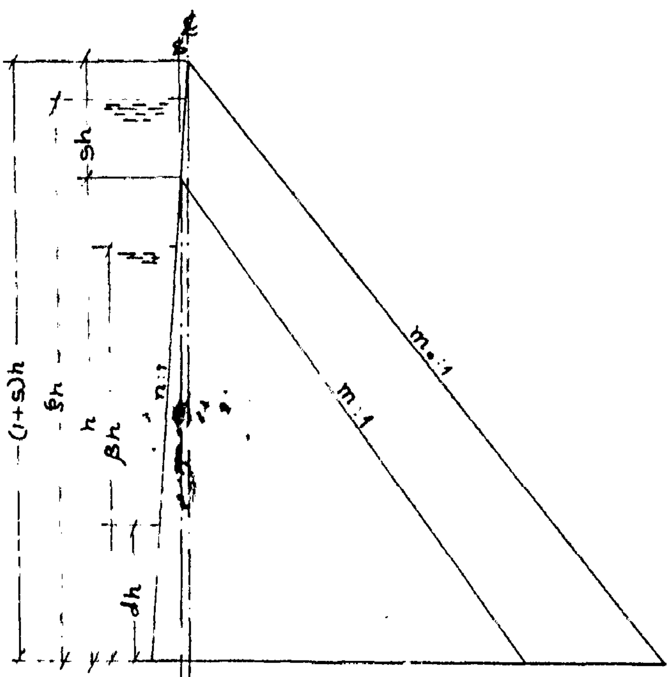



FIGURE - 3·10

TYPICAL SECTION CONSIDERED BY JAPANESE ENGINEERS IN CONNECTION WITH DETERMINATION OF GENERAL EQUATION FOR D/S SLOPE OF BACKING 'm'.

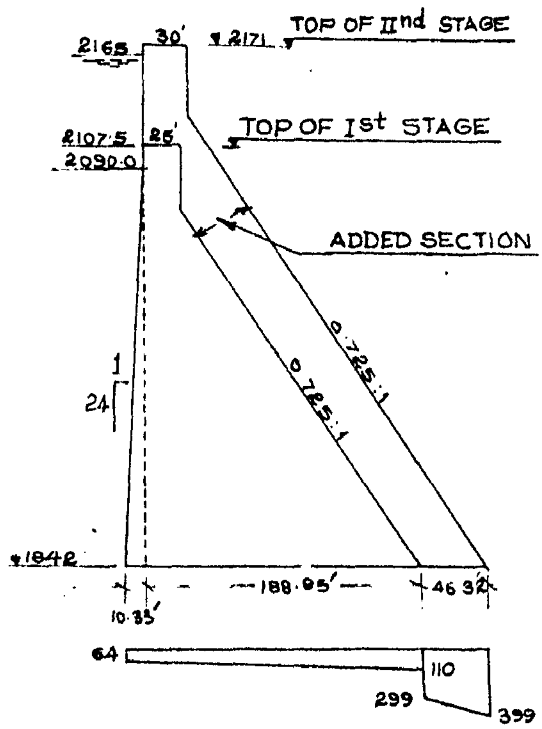



FIGURE - 3·11

STRESS DISTRIBUTION AT THE BASE OF KOYNA DAM IF IT WERE STRENGHTENED AND HEIGHTENED AS PER EARLIER PROPOSAL.

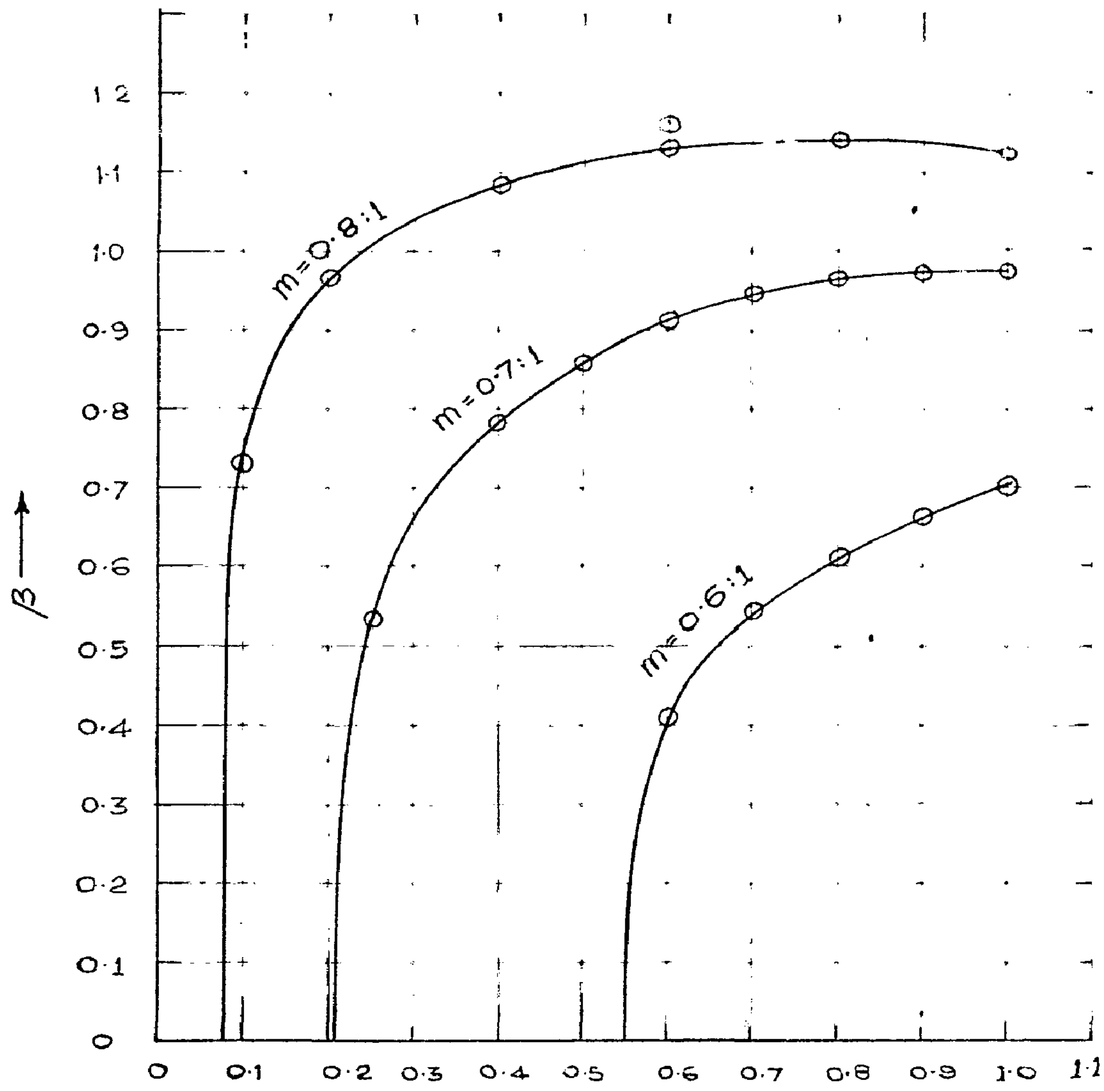
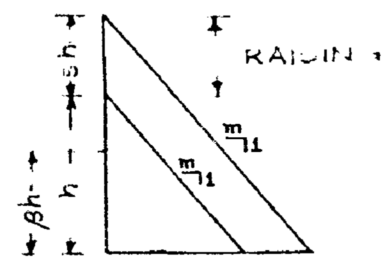



FIG. 3·12

RELATION BETWEEN 'β' & 'S'




KEY-DIAGRAM

# CHAPTER IV

# CONSTRUCTION CONSIDERATIONS AND TECHNIQUES IN STRENGTHENING GRAVITY DAMS

# CHAPTER IV

## Construction considerations and techniques in strengthening Gravity Dams :

### 4.0 General

In the previous Chapter, the design considerations involved in the raising and strengthening of gravity dams have been discussed. It is now proposed to discuss those special construction features and suitable construction procedures, which are necessary to be adopted to attain fulfilment of the design conditions.

### 4.1 Initial separation of new section from the old:

4.1.1 It is necessary to keep the new concrete mass separate from the old concrete mass initially from two considerations :

i) To allow movement of the new concrete to take place without setting up severe shearing stresses at the junction of the two masses and to check the tendency of the new concrete to crack in unpredictable directions,

& ii) to keep down the rise in temperature in the new concrete as low as possible and also to prevent heat transfer from it to the old wall (23).

4.1.2 The new concrete can be kept separate from the existing section in three ways as follows :-

a) The new section may be separated from the old section through a system of mild steel rods.

b) The new section can be kept separate from the old dam by resting it on the downstream slope of the existing dam through precast ribs.

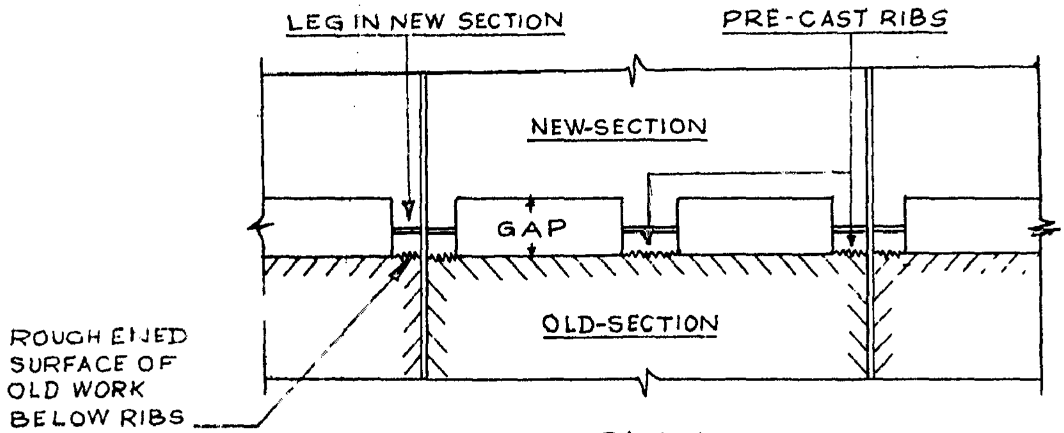
c) The new concrete may be separated from the old section by lacing it against precast slabs with legs resting on the downstream face f the existing section.

.1.3 In the first method, holes are drilled on the downstream face of ne existing dam in a direction at right angles to its downstream face, the epth of these holes being dependent on the consideration of bond for ransfer of loads. Mild steel rods are inserted in these holes and routed. The rods are of sufficient length so as to span across the idth of the gap proposed to be kept between the old and the new sections nd to get embedded into both the sections for adequate length. In the ase of Asswan dam in Egypt, the rods were embedded for 1.5 metre length n each section. Flexibility of these rods over their free length in the idth of the gap is sufficient to allow unrestrained movement of the slab uring its shrinkage and settlement. At the same time the rods are strong nough to offer adequate reaction to support the new mass on the old ection (31). The rods are well embedded on both sides so that dependence s not placed on their terminal points alone transmitting pressure, but dvantage is taken of their bond as well, thus making certain that there s no possibility of movement in the direction of their length. The epth of embedment provided should be adequate from this consideration . t may be that the two sections have a tendency to move away from each ther during an earthquake shock in which case these bars adequately mbedded in both the sections will serve to prevent occurrence of such a eperation. An example of this method of supporting the new backing is he first heightening of Asswan dam in Egypt. In this case, the original eight of 12.4 metres was raised by 5.0 metres by adding a stone masonry acking of 6.18 metres thickness on the downstream side. Steel bars of 2 mm diameter, one per square metre of the surface area, were provided or supporting the new work. A minimum gap of 15 cms width was kept etween the new and the old work. This gap was not grouted for 2 years

ill the new mass attained stable temperatures as in the existing
ork (11,13).

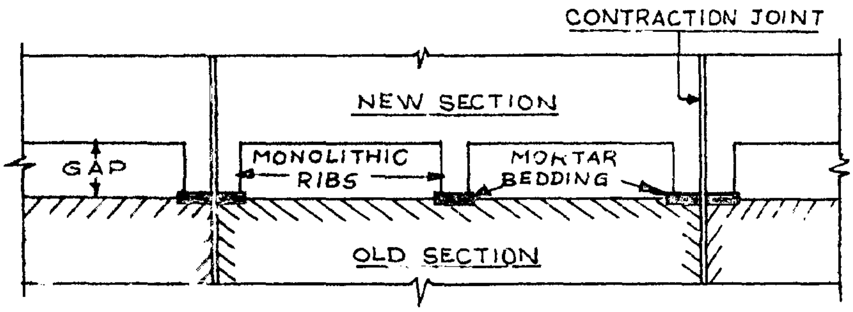
.1.4 In the second method, ribs resting on the downstream face of ne old dam are provided on which the new concrete section rests leaving clear gap between the downstream face of the existing dam and the nner face of the new concrete section in the remaining portion. Generally nree ribs are provided, two out of which are located adjoining the ransverse contraction joints in the old work, i.e. at either end of each onolith, the third rib being provided centrally. It may be mentioned ere that, the transverse joints are continued in the new work in the ame vertical plane as in the old work when full backing is provided.

.1.4.1 There can be a variation in the provision of such ribs. These ibs may be cast on the downstream face of the old dam the latter having een roughened to create good bond between the two. The new backing an then be made to rest on these pre-cast ribs through corresponding egs formed while placing it. This arrangement is shown in plan in the igure below. This method was used in the construction of the strength- ning section for the Alpine dam in California (2,34).




PLAN

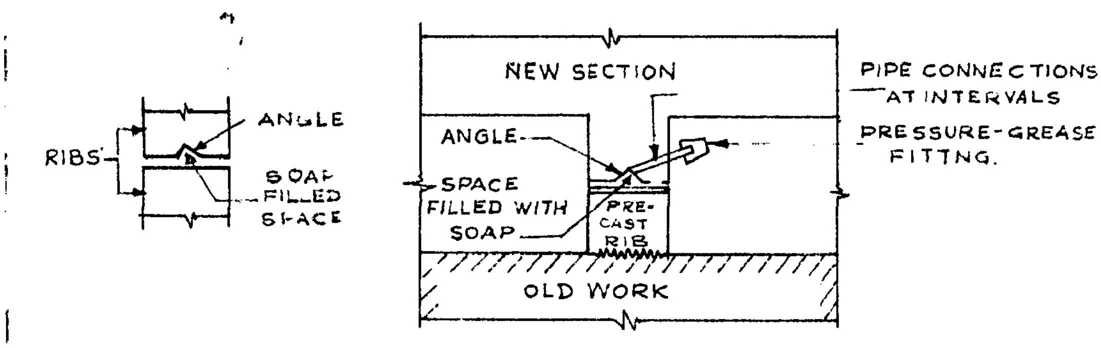
4.1.4.2 An alternative method is to chip out on an average 2.5 cms thick concrete on the downstream face of the existing structure and to form a bedding of cement mortar of 5 cms thickness under the ribs. The surface of this mortar bedding is trowelled to a smooth finish. On this bedding is made to rest the rib formed monolithic with the new concrete backing in such a way as to leave a gap of the desired width between the two works. This is shown in the figure given below. This method was adopted in strengthening of Mundaring weir in Australia (23), Mullardoch dam in Scotland (21), O'shaughnessy dam in San-Franscisco (14) and Lages dam in Brazil.




4.1.4.3 When the new work is supported on the old work through ribs, the surface of contact between the precast ribs and the legs of the new section resting on the ribs in the first method and that between the monolithic rib and the mortar bedding in the second method is made as smooth as possible to prevent bond and allow free sliding movement of the new mass along the joint. This prevents undesirable cracking of the new mass as by such an arrangement its movement is not restricted and therefore there is generally no development of undesirable stresses at the joint.

4.1.4.4 An interesting system was adopted in the case of Alpine dam in California (2) to obtain a smooth frictionless joint between the ribs and the legs of the new section butting against these. In this case, ribs of width 30 cms and height 20 cms were built vertically on the downstream face of the existing section at 6 metres centres. These ribs were faced

faced with 24 gauge galvanised steel sheets to prevent bond between the legs of the new section and the precast ribs. 25 mm x 25 mm x 3 mm angle was embedded in the butting face of the leg of the new section by placing the angle longitudinally on the face of the precast rib. At intervals, the angle was fitted with a small pipe connection which extended through the sides of the formwork for the leg and terminated in a pressure grease fitting. Just before pouring of the new section, the space between the angle and the sheet metal on the precast rib was filled with liquid soap by operating a soap filled lubricating gun on the grease fittings. During the period of shrinkage of the new section, the liquid soap in the sliding joints of the ribs ensured freedom of movement without inducing any stresses. The figure given below shows the arrangement.




ARRANGEMENT USED ON ALPINE DAM TO PROVIDE A FRICTIONLESS JOINT DURING CONSTRUCTION.

This method has been extensively used in the strengthening operations of dams.

4.1.5 The third method of seperating the new concrete from the old dam consists of placing precast concrete slabs of suitable size on the down-stream face of the existing section. These slabs rest on the surface of the dam on their legs so as to form a gap of the desired width between the dam and undersides of these slabs. This gap is generally 15 cms in width.

The precast slabs are provided with lugs at four corners and keys on either side to form bond with concretes on either side. The slabs are suitably reinforced to carry the load of the concrete. Their surfaces also are kept rough so as to bond well with concretes on either side. 

To permit the new concrete to contract and settle down, the horizontal joint lines between the successive layers of these slabs are provided with sealing strips of rubber of adequate thickness to permit free contraction.

The lugs and end sealing strips of the slabs can be made to rest on specially provided bituminous pads firmly fixed on the concrete of the old dam to permit free movement of precast slabs with reference to the old dam. The lugs and end sealing strips of the slabs have to be provided with rounded and smooth contact surfaces to permit easy movement.

Holes have to be left in these slabs for insertion of dowel bars when provided.

This method was used in the strengthening of Barker Dam. Slabs of size 2 m x 3.66 m (length) x 0.2 m (thickness) were used (2). The arrangement has been shown in figure No.4.1 . This arrangement has not been much used in case of dams strengthened so far.

4.1.6 The section to be added on the downstream may not need support from the existing section for its stability, if its unsupported height is kept within certain limits. Thus it may be possible to construct the section to be added for reasonable heights - 8 m to 12 m - without any support, if the slope of its overhanging surface is not much flatter than 0.7 H:1v. In case of strengthening of Koyna dam, this was done, the use of dowel bars across the contact joint between the two sections being solely for resisting shear & tensile stresses likely to develop at the joint. Thus use of ribs can be dispensed with.

4.1.7 Comparative Merits & Demerits :

In the first two methods, the underside of the backing requires the use of costly formwork and its supporting arrangement. When dowel bars are to pass across the formwork, this cost is further enhanced. In the third method, precast slabs which get ultimately embedded in the work serve as the formwork for the backing concrete at the time of its placement and hence there is an appreciable saving on shuttering costs. Also the operations of hacking the underside of the backing concrete to obtain good bond at the joint of the backing concrete and the gap filling concrete which are required to be carried out under difficult conditions in the first two methods are eliminated automatically in the last method. However the advantage of visual inspection and control in proper filling of the gap without leaving voids and weaknesses is available only in the former two methods. In the third method one has to rely on the hope of getting a properly filled joint by grouting in the most workmanlike manner.

In some instances, the backing section has been constructed seperate from the old section with the provision of a seperation gap of 1.2 metres and without the use of ribs for support. In such cases & especially when strengthening is effected by the provision of downstream buttresses, the underside of the backing section may provide a comparatively more effective cooling surface due to better air-circulation facility around the backing section on all its faces. This may reduce the cooling time thus effecting savings in the cost of operating the cooling system.

In case of the work of strengthening of Koyna dam, use of ribs for supporting the downstream strengthening section was not made.

4.2 Precautions to obtain good bond at the joint

After the new concrete has cooled sufficiently and attained stable temperatures, it becomes necessary to grout the gap to obtain monolithic structural action. In order to obtain a strong bond between the old and the new sections ensuring thereby monolithic action of the structure as a whole, it is necessary that the bond at the joint is made strong enough to successfully resist the shear stresses developing at the joint.

The various measures generally adopted to increase the bond strength at the joint are :-

i) Roughening of the old concrete surface by chipping manually or with pneumatic tools for a depth of about 12 mms to 20 mms ($\frac{1}{2}$"-3/4").

ii) Cutting shear keys in the downstream face of the dam and forming them on the upstream face of new backing concrete at the time of its casting.

iii) Providing dowel bars to assist in resisting shear stresses.

4.2.1 Roughening of the concrete surfaces of the old and the new work is an arduous and a costly job. The intention is to expose a fresh, compact and a sufficiently rough concrete surface to effect a better bond at the joint. To achieve this purpose it is necessary to do the chipping only a few days in advance of the placement of concrete against it. The chipping of the sloping and hanging underside of the new concrete is more difficult than the hacking of the surface of the old section. Thus although the job is of a comparatively simple nature, it is time consuming and is required to be executed under difficult conditions.

The roughened surface of the concrete is required to be kept wet for about 3 days in order to prevent absorption of water from the new concrete placed in the gap. It is also necessary to see that, the surface is properly cleaned by air water jets and then a layer of cement mortar about 2.5 cms to 5 cms thick is applied over this surface just before placing concrete in the gap. Similarly thorough vibration of concrete near the contact surfaces on either side of the gap needs to be ensured. Such a procedure will prevent formation of voids at the joint which otherwise may weaken its strength. Utmost care and vigilance is necessary in controlling this phase of the job.

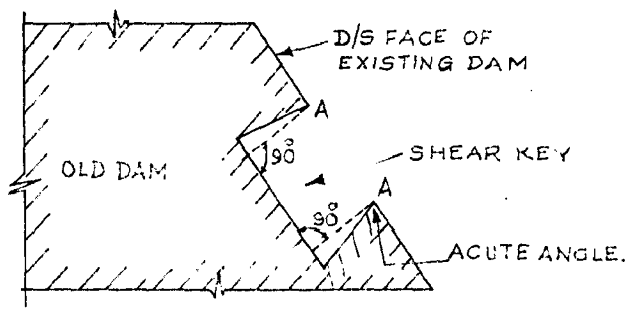
4.2.2 When it is expected that, the bond obtained due to placement and consolidation of fresh concrete in the gap after proper roughening of the joint surfaces will not produce the required strength to resist the shearing stresses that may develop at the joint, provision of dowel bars and shear keys is made to assist in resisting these stresses. The shear keys are also useful in effecting transfer of stresses across the joint (in a direction not normal to the joint plane) in case the slot concrete undergoes shrinkage so as to form small seperation voids at the joint.

4.2.2.1 When shear keys are provided, a crack will be developed at the joint only when the concrete forming the keys is sheared off in the plane of the joint. The force required to shear off the concrete is definitely more than that to break the bond of two concretes which have been just placed one over the other. Provision of shear keys therefore increases the overall strength of the joint in resisting shear stresses. These have been provided in the case of Koyna dam in Maharashtra, Ross dam in U.S.A, Alpine dam in California and O'shaughneny dam in Sanfranscisco.

4.2.2.2 The keys are in the form of rectangular or trapezoidal notches cut in the old concrete or formed on the underside of the backing concrete. Cutting shear keys in the concrete is a very costly operation. Cost on this account may be of the order of Rs.80/- per cubic metre (1973).

Provision of such keys not only involves excavation of set concrete but also filling of the notch by new concrete. Thus provision of shear keys is a costly affair but has to be provided when stresses of high magnitude occur in the region of the joint.

4.2.2.3 The excavation of these nothces is carried out by pneumatic breakers. Utmost care is required while cutting concrete near the edges of the keys to obtain a right angled cut. A rounded corner is not desirable. Some designers favour the use of the following shape of the key when the structure is located in an earthquake region.




By giving such peculiar shape, the shear key is expected to ensure proper interlocking by wedge action and thus to help in preventing separation of the old and the new sections if they try to do so in an earthquake shock. However the acute angled sharp edges at the locations 'A' shown in the above figure are difficult to obtain during the excavation of the key.

In some instances, when the stresses at the joint are low, shear keys of only 7.5 cms depth have been provided on the upstream face of the new hanging section as a substitute for roughening this surface, the latter operation having been found difficult to execute (23).

Example of Mundaring Weir in Australia can be cited in this respect. Please refer figure 4.7.

4.2.3 Dowel bars can be provided with either of the following objectives :-

i) To provide support to the new concrete during construction before filling the gap.

ii) Prevention of shrinking away of new concrete with respect to the old.

iii) To assist in resisting shearing stresses at the joint.

4.2.3.1 Their purpose in supporting new concrete during construction has been already discussed. However it may be added in this connection that, some authorities appear to prefer the method of using ribs to support the new section when the addition is massive (23).

4.2.3.2 The dowel bars will also serve the same purpose as the shear keys during an earth tremor. So also they will prevent the shrinking away of the slot concrete from its joints with the old and the new concretes thus ensuring a tight joint.

4.2.3.3 When shearing stresses of high magnitude as well as tensile stresses are expected on the joint plane, the use of dowels can not be avoided. The use of dowels in large quantity in the case of strengthening of the Koyna dam has been made on this account.

4.2.3.4 Provision of dowels involves the costly operations of drilling holes in the old masonry or concrete to the extent of 1.5 metres to 2.4 metres (5' to 8') depth depending ~~the~~ upon the purpose of their provision. These holes have then to be cleaned and filled with cement grout to fix the anchor bars effectively. The holes are cleaned by air and water jets to remove the dust from inside the holes formed during drilling as well as to remove any foreign matter and loose particles which remaining inside can affect bond length necessary for effective anchorage. The holes have to be kept wet for about 2-3 days before fixing the dowels in them.

4.2.3.5 Percussion drilling is normally used for this purpose. It is necessary to ensure that, the holes are drilled exactly normal to the downstream face of the existing dam. The diameter of these holes is generally kept about one and a half times the diameter of the anchor bars.

4.2.3.6 Proper procedure to completely fill the annular space around the bar during grouting is necessary to make the length of the bar for the full depth of the hole effective in offering resistance to its being pulled out. The correct procedure in this respect is to fill the hole completely by cement grout of as thick a consistency as is possible to use, as a first operation. This may be done by inserting a pipe in the hole right upto its bottom and pouring cement grout through a funnel into the pipe. The pipe is gradually withdrawn as the hole fills with the grout. After filling the hole nearly upto its brim, the bar can then be gradually pushed inside the hole to displace the excess grout. It is also necessary that all precautions are taken to ensure that these bars once anchored are not disturbed until the grout in the hole has attained its final strength. So also every care needs to be taken to ensure that, the bars are not bent until they get embedded in the new concrete. The bars are preferably given a coating of cement slurry just before inserting or embedding them.

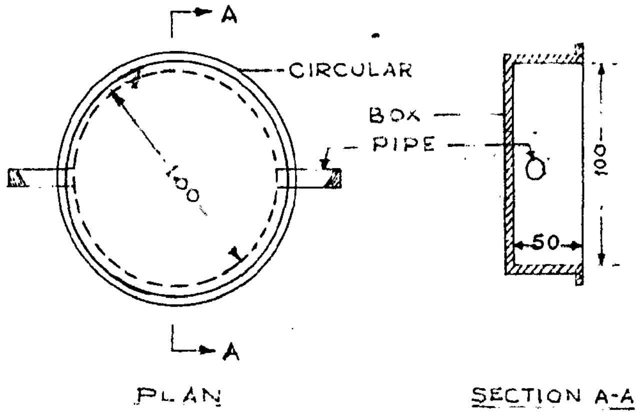
4.2.3.7 The shuttering arrangement needs to be made to suit the spacing of the dowel bars. From this point of view, a 1.5 metre (5') vertical spacing of these anchor bars alongwith the adoption of 1.5 metre (5') thickness of lifts of new concrete is very convenient.

4.3 Grouting Arrangement

4.3.1 This is necessary to grout any shrinkage gaps which may form due to separation of gap concrete from the downstream face of the existing dam and the underside of the new concrete section. The grouting system is therefore generally provided on both the faces of the slot.

4.3.2 It is, however, felt that, as the downstream slope of the existing dam section is generally flatter than 0.7:1, the slot concrete resting on it should not have a tendency to shrink away from this face. Hence it is felt that, the provision of costly grouting system on the downstream face of the old work need not be considered as obligatory. It is however generally provided for filling possible voids at the junction between the slot concrete & the downstream face of the old section, in case they exist due to reasons of inadequate vibration of concrete near the contact surface.

4.3.3 The system generally consists of black steel pipes of 2.5 cms (1 inch) internal diameter placed on the surface of a monolith horizontally and spaced vertically at suitable intervals. These are fed through a supply pipe of about 3.75 cms ($1\frac{1}{2}$ inch) diameter placed on the surface of the monolith near and parallel to one of its contraction joints and along its downstream slope. A return pipe of the same size is placed near the other contraction joint of the same monolith. Please refer figure 4.2 in this connection. On the horizontal grouting pipes, grout outlets (also called grout buttons) are provided at suitable intervals to allow the grout to escape out and permeate into the shrinkage gap. The grout outlets are nothing but circular boxes of about 10 cms diameter and 5 cms height made out of galvanised iron sheet with two pipe connections on either side as shown in the following sketch:




GROUT-BUTTON

4.3.4 The portion of the roughened surface over which these buttons are to be placed is plastered with a thin coating of cement mortar to prepare a surface on which the sides of the box can rest properly. The surface of the box which is to rest on this mortar seating is applied with grease to prevent cement mortar entering inside and blocking the passages for the escape of grout at the time of grouting.

4.3.5 It is necessary to exerdse care at the time of vibration during concreting near these grout outlets to ensure that they are not disturbed once kept in place; otherwise the very purpose of their provision will be defeated.

4.3.6. In addition to the grout buttons on the plane surface of the interfaces, it is necessary to provide grout buttons inside the shear keys also.

4.3.7 Grouting is under-taken after the completion of shrinkage of the gap concrete. Grouting at low reservoir stages is preferable. Grouting pressures used are of the order 1 to 2 kg/cm$^2$. Large pressures will have the tendency to cause separation at the joint.

4.3.8 In the case of Burrinjuck dam in Australia, an analysis of the amount of the cement injected at the two interfaces showed that, if all cement was used in filling the shrinkage space, a total shrinkage of the 0.6 metre thick gap concrete would average to about 0.5 mm (32). As this concrete was, however, placed by pumping and pneumatic vibration, probably all voids near the interfaces were not filled and so some cement may have been used in filling these voids. The time of grouting at different elevations was related to lake levels on the same lines as is done for the placement of the gap concrete.

4.4 <u>Use of Adhesives</u>

4.4.1 To increase the strength of bond at the joint, adhesives in the form of resins such as for instance epoxy resin can be applied on the roughened surface at the time of placing new concrete in the gap. This resin after hardening becomes even stronger than concrete.

4.4.2 However these resins are extremely costly to use in large quantities as in the present case under consideration. Also short setting time of these resins imposes a restriction on their use when large surfaces have to be treated, as application to the old surface and the subsequent operation of laying new concrete against it will have to be done within this period which is difficult indeed. Also the long term effects of introducing a foreign material such as epoxy resin inside concrete is also not yet fully known. Epoxy adhesive is reported to have been used for obtaining a good bond at the junction plane while strengthening a concrete gravity dam of Hauser development in U.S.A. in the year 1969-70 (48).

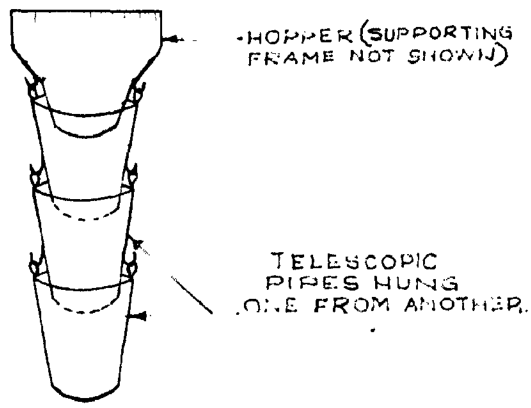
4.5 Method of Gap Filling

4.5.1 We have seen earlier that, an air gap is kept between the old dam and the new section initially. The width of this gap or slot may vary from 15 cms to about 1.2 metres depending upon the method of construction adopted for supporting the new section. Thus when the new section is supported either on anchors embedded in the body of the old work or on ribs resting on the downstream face of the existing section, the gap or slot is generally of 90 cms. to 120 cms (3 ft. to 4 ft.) width. When precast slabs are used, the slot width is about 7.5 cms to 20 cms (3" to 8").

4.5.2 When the gap or slot is of 15 cms width, as when precast slabs are used, the slot is filled with pre-packed concrete. However when it is of 90 cms to 120 cms width, normal concrete is generally used for its filling. This gap is filled when the new concrete addition attains stable temperature conditions.

4.53 The placement of gap concrete is done at appropriate reservoir elevation (Refer para 3.2.5). It is also necessary to control the temperature of the gap concrete. This gap concrete will be best used if it is made to act like a tight plug. The idea is to control its placement temperature as well as its temperature rise to such an extent that its maximum temperature reached during the setting process shall be less than the temperature of the concrete in the old dam as well as that of concrete in the new section surrounding it. Such a temperature condition shall make the gap concrete expand further and remain in that state, devoid of any tendency to shrink, thus butting tightly against the surfaces on either side. In such a case, grouting will also not be needed. Such an ideal control will of course not be always possible and that too at all sites, as climatic conditions will be one of the important factors involved.

4.5.4 Placement of gap concrete through a large height in a slot which is 90 cms to 120cms wide with reinforcement across it presents problems of restricted working space and saggregation. The concrete is normally chuted down through a system of telescopic pipes loosely connected to one another as indicated in the following sketch :




ELEPHANT TRUNK FOR POURING CONCRETE.

When the width of the slot is less than 0.9 metre or so, the above metho probably becomes difficult to use. In such a case, if concrete alone is desired to be used, pumping the concrete into the gap may have to be resorted to as was done in the case of Burrinjuck dam in Australia.

4.5.5 It is not convenient to use mass concrete for the slot portion due to this system of placement. The larger particles in the case of mass concrete may block the tapering end portions of the pipes and create difficulty in continuous placement. So also the advantages in the use of low slump concrete having big size aggregates cannot be availed of in this location as the concrete will have to be necessarily of much more fluid consistency to enable it to flow down the series of pipes without sticking. Also working of concrete in the restricted space is much better with concrete having lesser maximum size of aggregate. Concrete with 80 mm as the maximum size of coarse aggregate is therefore preferred for gap-filling.

4.5.6 While concreting in the restricted space in the slot, care is necessary to see that the corners in the shears-keys are properly filled. So also the embedded parts such as grout buttons, reinforcement, embedded instruments must not be disturbed in the vibration operations carried out to consolidate the concrete. This operation is thus very difficult and therefore has to be done with due care.

4.5.7 Temperature control of the slot concrete is particularly important especially as regards its placement temperature, as it has no free face to radiate out the liberated heat. The lower the placement temperature, the lower is the maximum temperature reached and hence lower is the temperature drop to the final stable temperature during cooling. In case of cooling of slot concrete, there is much less likelihood of steep temperature gradients up in it due to the absence of effectively large free cooling surfaces in contact with the relatively much cooler atmosphere. The main loss of heat is by conduction to the surrounding concretes and as the thickness of the slot is just about a metre or so, steep & differential temperature gradients will not exist within the mass. So cracking at the joint on account of steep temperature gradient is not possible. Cracking is thus only possible due to large temperature drop in cooling, which can be avoided by properly controlling the placing temperature and if necessary by post cooling methods.

4.5.8 <u>Pre-packed concrete for slot filling</u> :

4.5.8.1 Pre-packed concrete was used in the case of Asswan dam in Egypt(11) and Mullardoch dam in Scotland (21) for filling the gap.

It involves placement of either gravel or crushed aggregate in the slot which is later on grouted by pumping cement grout into its voids. One of the main advantages in case of such concrete is appreciable reduction in drying shrinkage. This is because the aggregate particles are in mutual contact before grouting and remain so afterwards. Whatever tendency to shrink is there in the grout, the same is resisted by compression between the particles of aggregate and therefore there is little drying shrinkage of the whole mass. As per J.Waddell(43) the drying shrinkage may thus be reduced to the extent of 50% to 60% when compared to that of the ordinary concrete. This is an advantage.

4.5.8.2 The size of the coarse aggregate to be used for filling of the slot depends upon the width of the slot. Generally the maximum size can be as large as is convenient to handle, provided that it does not exceed one fourth of the width of the slot. The minimum size may not be smaller than 12 mm which is used when the slot width is less than 305 mm or 18 mm when the slot width is larger than 305 mms.

In case of Mullardoch dam in Scotland, the aggregate was graded from 87 mm to 62 mm, the width of the slot being 0.90 metre. In the case of an alternative of prepacked concrete joint considered for the strengthening of Koyna dam (50), the size of aggregate considered was 10 mms to 18 mms for 75 mm width of slot and 10 mm to 40 mm for 150 mm width of slot.

4.5.8.3 The aggregate can be placed in the slot simultaneously as the new backing is independently being raised. Extreme care is necessary in such a case to prevent any outside material, dirt, dust etc. falling into this filling and getting mixed with it. In addition, the cement slurry from the backing concrete must not be allowed to flow inside this material. The gap therefore needs to be kept fully and properly covered after placement of gravel or crushed metal inside it. This procedure was followed in the case of Asswan dam.

4.5.8.4 If the aggregate is not filled in the slot simultaneously with the raising of the new section, its placement at a later date presents problems. This procedure is of course not possible when precast slabs are used to serve as formwork for the upstream face of the new section.

In the case of Mullardoch dam, where the new section was supported on ribs resting on the old-section and filling of the gap by aggregate was attempted later, it was feared that, the stones if let hurtling down the whole depth of the slot may get shattered and may form pockets of dust in the aggregate mass making proper penetration by the grout difficult. The method then adopted was to pour the grout into skips containing the aggregate and tip the mixture into the slot through elephant trunking (21). The mixture, when it arrived at the bottom was spread and compacted by hand. The grout used was so thin that it did not prevent direct contact between the aggregate particles and thus although the usual method for pre-packed concreting was not followed, it is claimed that the advantage of reduced shrinkage was still not lost. The thin grout, in addition, improved the compaction of the aggregate by acting as a lubricant. This method, although it was a forced solution to the problem, also facilitated control of the moisture content of the aggregate before adding grout. The grout consisted of 1 part of cement to 3 parts of sand with a water cement ratio of 0.90. The rate of filling was limited to 1 metre/hour to avoid hydrostatic pressure acting on the downstreamm mass.

4.5.3.5 In the case of Asswan dam, 68 mm dia. galvanized wrought iron pipes were kept in position when the slot was filled with aggregate. These pipes were placed along the downstream slope of the old dam at 7 metres centre to centre. These were perforated with 20 mm (¾") diameter holes placed hit-and miss on the opposite sides of the pipe, one hole being provided for every 23 cms(9") length of the pipe. Mixing machines of 10$m^3$ capacity each were used for supply of grout. The grout was delivered from each machine through three taps, to which was attached flexible steel piping 50 mms in outside diameter. These flexible pipes were dropped down the embedded 68 mm diameter grout pipes to such a level that the grout had to flow from the end of the flexible pipe, then through the holes in the main pipe into position without thus falling down through any length of the 68 mm diameter pipe. The grouting was carried out for a height of 7 to 8 metres at a time to prevent hydrostatic pressures on the new backing (13). However, some prefer continuous grouting for the entire depth of the slot in one stretch, as in their opinion setting of the grout takes place generally in an hour or so and hence it is not necessary to take into account hydrostatic pressure effect (23). The grout proportion used in case of Asswan dam was 1 part of water to 1.2 parts of cement by volume, grouting being done in the cold season.

4.5.8.6. When precast slabs are used for supporting the new section, grout stops consisting of 16 gauge galvanized iron plane sheets with a 'U' shaped kink are provided to allow the grouting to be carried out in vertical stages of about 7.5 metres. The end slabs in monoliths have about 150 mm thick concrete sealing strips to confine the lateral spread of the grout and to keep the contraction joints free of grout. Tell-tale pipes are provided at intervals to indicate the travel of grout and also to serve as air-Vents. Grouting is done through a system of 25 mm or 40 mm. diameter conduit pipes perforated suitably along their lengths. Before commencing grouting of any particular stage, fresh water should be circulated through the grouting system of pipes to check that all passages of grout are free from chocking. The arrangement which was proposed in the case of strengthening of Koyna dam is shown in figure 4.1.

4.5.8.7 The grouting pressure generally used is of the order of 2 kgs/cm$^2$. Grouting is done from the bottom of the stage upwards.

Some times an intrusion aid is used to increase fluidity of the grout. Aluminium powder is also sometimes added to give some times added to give some expansive effect to the grout before setting.

4.6 <u>Temperature Control</u>

4.6.1 As seen earlier, it is necessary to keep the temperature rise in the concrete as low as possible to prevent cracking. So also as the new backing is required to be joined with the old section only after the former has attained stable temperature conditions, it is desirable to reduce this cooling period of the backing concrete as much as possible to avoid idle periods. Without temperature control, this period will depend upon the thickness of the backing concrete to be added. In case of Mullardoch dam in Scotland, the thickness of backing slab was 2.70 metres and the backing slab was allowed 5 months to take up its normal shrinkage.

In case of Asswan dam, the thickness of masonry added was about 6.1 metres (horizontally). The period of cooling allowed was 2 years.

In case of Mundaring weir, the thickness of the backing was 15.25 metres. The construction of the new section, which was of about 36 metres height, was planned in three sections for adequate dissipation of heat. Thus raising of the bottom 8.55 metres was followed by a break of 6 months; further raising by 21.4 metres height was again followed by a break of another 6 months. 4 months' time gap was allowed between the completion of the last portion of the height and the commencement of slot filling (23).

Owing to the great importance of temperature control in the construction of the strengthening work, a discussion of the various measures adopted for temperature control will not be out of place.

4.6.2 Problem of cracking associated with heat generation can be considered to be two fold (when mass concrete is used for backing) -

(i) Thermal cracking results when there exists a large difference in temperature between concrete at the surfaces and that in the interior of the mass of concrete due to differential cooling.

(ii) During cooling to final stable temperatures, the decrease in size may cause cracking if the mass is restrained from change in volume due to restraint offered by the adjacent concrete or rock foundation.

Thus temperature control is necessary to prevent concrete from undergoing drop of temperature greater than that which it can withstand without cracking.

The only solution is to restrict its maximum temperature. This can be done by resorting to the following measures -

a) Proper design of the concrete mix,

b) Selection of cement or other cementitious materials,

c) Precooling of concrete ingradients,

d) Control of the lift thickness and time interval between lifts,

and e) Post cooling.

4.6.2.1 <u>Proper design of the concrete mix</u>

As heat generated in setting is proportional to the quantity of cement per unit quantity of concrete, proportioning of mix may be done so as to keep down requirement of cement to the minimum. Particular care needs to be taken in respect of the following -

i) Use of the largest practicable aggregate size reduces cement content. Problems of handling and mixing however may limit the maximum size which can be used.

ii) The mass concrete shall always be air-entrained. This reduces requirement of water for producing the desired workability without affecting the other desirable properties of concrete. This in turn reduces cement content. Use of entrained air can reduce the cement content by 25 kg/ $m^3$ in case of mass concrete. Temperature rise in concrete gets reduced by about 20% as a result. Air content generally prefered is 3% to 4% (41,42).

iii) Consistency : Adoption of as stiff a consistency as possible results in low water content thus reducing cement content. The stiffer the consistency used, the better should be the control on aggregate graduation and accuracy of batching to obtain uniform quality concrete.

4.6.2.2 Selection of Cement

This is however many times not possible due to non availability, high cost etc. In such a case it is better to determine the adiabatic temperature rise of cement received from different factories and use the one giving the least rise. Some specifications in case of American dams require that, the cumulative heat of hydration shall not exceed 65 calories/gm @ 7 days and 80 calories/gm @ 28 days (Pine Canyon dam in California).

Use of pozzolans :

These can be used in mass concrete for achieving economy, decrease in heat generation (although slight), improved workability and reduced permeability. Fly ash (a fine ash which results from burning powdered coal in power plants), and natural materials like opaline cherts and shales, diatomaceous earth, tuffs, volcanic ashes, pumicites serve as pozzolans. Most of these require calcining to be effective.

These when effective can be used to replace upto 35% of cement content in mass concretes. They should comply with specifications given in IS 3812-1966 (Part I) when use of fly ash as pozzolana is desired & IS 1344-1959 when use of burnt -clay is to be made. Chief difficulties in their use can be high mixing water requirement when natural materials are used and high carbon content when fly ash is used. The latter increases the requirement of air entraining agent and makes control of air content difficult. Pozzolans react with lime formed during hydration of cement to form strong hydration products (42). Low early strength results but their use means economy and reduction in early heat generation potential.

4.6.2.3 Precooling of Concrete ingradients :

Maximum temperature attained by mass concrete depends on the following :

(i) Initial placing temperature,

(ii) Adiabatic temperature rise potential of cement used,

and (iii) Heat gained or lost from concrete during the period of hydration.

Control of maximum temperature through control of initial placing temperature of concrete will be discussed in this paragraph. Control of initial placing temperature of concrete can be done by cooling the ingradients of concrete before mixing. Coarse aggregate occurres in the largest quantity in a concrete mix, while water has the highest specific heat among all the components of concrete. These two ingradients at the same time are also amenable to precooling.

Generally cooling of coarse aggregates is a costly process and difficult due to low specific heat of stone. Cooling of water is better as its chilling can be conventiently achieved by conventional refrigeration methods or by use of ice.

In arid climates, sprinkling of storage piles can be done to augment other cooling methods.

It can be said that on an average, concrete can withstand a sudden drop in temperature of about 20°F and a gradual drop of about 40°F to 60°F. Temperature control is concerned with obtaining a gradual drop when the total drop exceeds 20°F.

If by precooling, temperature drop could be satisfactionaly controlled by restricting the peak temperature reached after hydration, post cooling which is costly is not needed.

4.6.2.4 Transfer of heat from concrete during the period of cement hydration which governs the maximum temperature attained by concrete after placement, can be considered under the following heads :

a) Lift thickness,

b) Time interval between lifts,

and c) Post cooling.

(a) Lift thickness:

This is determined from economic znd technical considerations. Thus reduction of thickness means increased expenses on lift-joint-cleaning. When concrete is not precooled below ambient temperature, thinner lifts are preferred as it helps to reduce the peak temperature due to radiation of heat from their top surfaces to the atmosphere. On the contrary, when concrete is precooled thicker lifts are preferable from economic as well as technical considerations as the thicker the lift, smaller is the average rise in temperature attributable to heat inflow from the surface. In case of uncooled concrete, 1.5 metre is the maximum lift thickness that is used. In this case, maximum temperature is reached in 3 days after placement. Increase of thickness beyond 1.5 metres makes it necessary to move concrete by vibration for an excessive distance at the bottom of the forms for the downstream face Practical upper limit can be regarded as 2.25 metres. Even when post cooling is used, lift thickness above 1.5 metres is likely to give rise to non-uniform temperature distribution within the concrete mass.

Thus 1.5 metres thick lifts are preferred for uncooled concrete or concrete containing cooling pipes, while 2.25 metres thickness of lifts is preferred for precooled concrete without post cooling. Figure 4.3 shows the effect of lift thickness on temperature rise within them. Special care, while placing concrete on irregular foundation and horizontal joints exposed for few weeks, is necessary. In such cases, the first two lifts of concrete are generally laid of 0.75 metre thickness with at least 5 days interval in between.

(b) <u>Time Interval between lifts</u> :

For precooled concrete, delay between placement of successive lifts is not preferable. It is desirable to cover it as quickly as possible by cool concrete. However with use of uncooled concrete, it is necessary to wait for certain period which may vary from 3 to 5 days, that is allowing 2 days cooling after maximum temperature is reached.

Excessively long exposures of concrete lifts are avoided to prevent too rapid a cooling and resultant possible cracking. Lifts are therefore covered preferably within 10 days.

Thus the temperature of placement can be calculated to be that which will not produce a peak temperature exceeding the final stable temperature beyond an allowable margin after taking into account the temperature raise due to the combined effect of heat generation by cement hydration and heat gain or less to the atmosphere and the surrounding concrete or rock in contact. In the calculations of gain or loss of heat, heat flow in the vertical direction only is generally considered for simplicity.

Such calculations can be made for different sets of conditions. Some typical results as given by Mr. Robert Philleo are given in figure 4.3. The bottom most graph in this figure demonstrates that the peak temperature may be obtained in a given lift when that lift is the top exposed lift or after it is covered depending on the heat producing potential of the cement. In case of low heat cements, the heat development in the early days is less, which explains the curve for low heat cement in this figure.

(c) Post cooling of concrete :

This is especially important when the thickness of the new section added for strengthening is so large as to result in inadequacy of precooling measures.

The chief advantage of pipe cooling is its flexibility. Cooling can be done to the desired extent, whereas with precooling this is not possible. Proper control of the system enables uniformity of temperature throughout the mass, thus minimising stresses developed during cooling. However if precooling alone is considered adequate, it is certainly more economical as compared to post cooling. Improper control of system may give rise to too rapid cooling causing cracking. Too rapid cooling, if done during early age of concrete may cause retardation of hydration of cement. In early ages, heat generation normally exceeds heat extraction by pipe cooling unless impractical pipe spacing is used.

## Design of Pipe Cooling System :

For this purpose the following must be known :-

i) Peak concrete temperature,

ii) Thermal diffusivity of concrete,

iii) Desired stable temperature,

and iv) Cooling period.

The following factors are determined :-

i) Pipe diameter,

ii) Pipe spacing,

iii) Length of each pipe coil.

iv) Water temperature.

v) Rate of flow through pipe.

i) Pipe diameter:

Its selection is generally the one which most economically passes the required flow of water through known length of the coil. Smaller diameter means lesser cost but greater pumping costs due to higher friction losses. Generally pipes of 25 mm external diameter are used.

Pipe diameter, however, has relatively little effect on cooling rate. Thus the effect of varying pipe diameter varies as the logarithm of the ratio of pipe spacing to pipe diameter.

ii) Pipe spacing :

Vertical spacing depends upon lift thickness. So horizontal spacing can be varied to obtain cooling within specified period. Horizontal spacing varies from 0.6 metre to 1.6 metres.

iii) Water temperatures:

Use of deep seated reservoir water can be economical as compared to the use of refrigerated water, when the former is available all the year round and its use can result in achieving cooling in the desired period.

iv) Rate of flow through pipe:

With refrigerated system the rate of flow is generally 4 g.p.m. or less. At higher flows, the increased cooling rate is not commensurate with increased pumping and refrigeration costs. When river water is used, flow rates as high as 15 gpm are used.

In case of Glen Canyon dam, water at 45°F was circulated through pipes during the period between 12 days to 30 days after placement. In the next month water at 35°F temperature was used.

v) Length of pipe coil

Since water gets warmed as it moves through the pipe, cooling is not uniform along the pipe length. Thus use of long lengths of pipe coils results in large difference between the mean temperature of concrete and the temperature in the vicinity of the outlet of the pipe coil. One method to avoid this is to provide interconnection between inlet and outlet headers to enable reversal of flow direction in the pipes. This also eventually helps to prevent clogging. The lengths of coils are therefore limited from thermal and hydraulic considerations to about 245 metres (800 feet), although lengths exceeding 400 metres have been used.

vi) Cooling period

Cooling operations should preferably be over before the concrete passes from plastic to the elastic state. In other words, volume changes in concrete due to cooling should be preferably complete before it looses its plasticity in which case it will not cause any damage as it will not induce any stresses.

In post-cooling, it has been experimentally confirmed that, the stress in concrete is relatively small in relation to the temperature change as the mass concrete is in a plastic state in its early age. On this basis, duration of cooling can be decided upon to restrict the tensiles stresses (which appear in the concrete at the transition period of changing from plastic to elastic state) below permissible limits. Such consideration will however be more important when the new concrete is being placed directly against the old section. Thus in case of Odomari dam, this period was assumed as 28 days (20).

The cooling period in other cases will depend upon economic considerations.

Figures 4.4, 4.5, & 4.6 as given by Mr.Clarence Rawhouser are useful in the design of pipe cooling system. Interrelationship of various factors is considered in the curves given in these figures. These curves are applicable to 25 mm outer diameter tubing with a vertical pipe spacing of 1.5 metres.

In framing these curves it is assumed that, no significant amount of heat is being added to the concrete by hydration during the period under consideration. If the concrete is gaining heat or inflow water temperature is not constant, these curves can be used in a step-wise computation. Mr. Rawhouser in his paper 'Cracking and temperature control of mass concrete' published in Journal of A.C.I., February 1945 has given the details of these computations.

vii) Following precautions are necessary when pipe cooling is used.

1) The pipes should be tested under static pressure to check for leaks.

2) Pumping may be done at design flow rate to check how friction losses compare with those calculated. Higher

losses indicate partial stoppage in the coil, which needs location and removal before embeddement.

3) Pipes are filled with grout after completion of cooling.

Embeddement of thermometers in the body of the backing at suitable locations is necessary to know the actual conditions of temperature within the mass. Comparison of this actual data with the estimated figures is then possible.

4.7 Drainage at contact joint

Drainage system is installed on the downstream face of the old section to take care of the water that may seep through cracks in the old section or through the joint in case it cracks. This thus helps to relieve uplift pressures which might develop at the joint in case of the above eventualities.

A typical layout, which was provided in the case of strengthening of Mundaring Weir is given in figure no.4.7 (23). Figure 4.8 gives details of drainage layout adopted in strengthening of a gravity dam of Hauser Development in America (48). Important features in case of Mundaring Weir are briefly explained below.

a) Galleries :

An upper gallery of size 1.15 mx2.1 m intersecting the junction between the two concretes is provided to pick up any water passing the horizontal seal across top of the dam.

A lower gallery of size 0.9 mx2m is provided in new concrete. It is laid to follow the natural valley slope. However in the lower part of the valley, it has been kept above the stilling pool level.

b) Vertical Drains:

These extend from the upper to the lower gallery along the downstream face of the old wall, their spacing being 1.9 m (6' 3") c/c. These are semi circular precast concrete pipes of 300 mm diameter. They are made to rest on prepared mortar bedding on the face of the old wall.

It is necessary to make the joints in these pipes grout-tight.

c) Horizontal drains :

Mortar bedding below ribs near contraction joints, on which the new section is supported, is likely to seal the contraction joints. To collect seepage from the contraction joints 100 mm diameter semi-circular pipes are provided horizontally at 1.8 m vertical spacing feeding into both the adjacent vertical drains. Thus two vertical drains serve each contraction joint for drainage.

d) Toe drain :

Toe drain of 0.6 metre square is provided at the downstream toe of the old wall. It is provided throughout the length of the old wall. Water in it is carried into the lower gallery through three 225 mm diameter relief drains.

Holes, 3 metres deep and 1.8 m c/c,have been provided deep into the foundation from the toe drain to provide relief from uplift.

e) Outlet drains :

450 mm diameter concrete pipes are provided connecting the lower gallery with the stilling pool for drainage. These outlets are provided with reflux gates to prevent backflow.

f) Horizontal Water Seal

This has been provided out of 450 mm high, 22 gauge Copper Plate interposed between the existing wall and the new concrete. It extends across the top of the dam and provides a water barrier at the joint between the existing section and the new section to be added above. A groove has been cut in the old concrete, seal placed in it and the groove backfilled with special dry mortar mix well rammed in place.

4.8 It will thus be seen that, the strengthening operations require much more care in execution as compared to the construction of a new dam. Peculiar construction features difficult of effective incorporation have to be provided, special nature of which results in an expensive construction. These expenses and safety of the structure on completion of its execution demand that, the construction Engineer carries out his work faithfully and with utmost care keeping in mind the necessity of realising the basic assumptions and considerations involved in the design of the entire structure assumed to act as one unit.

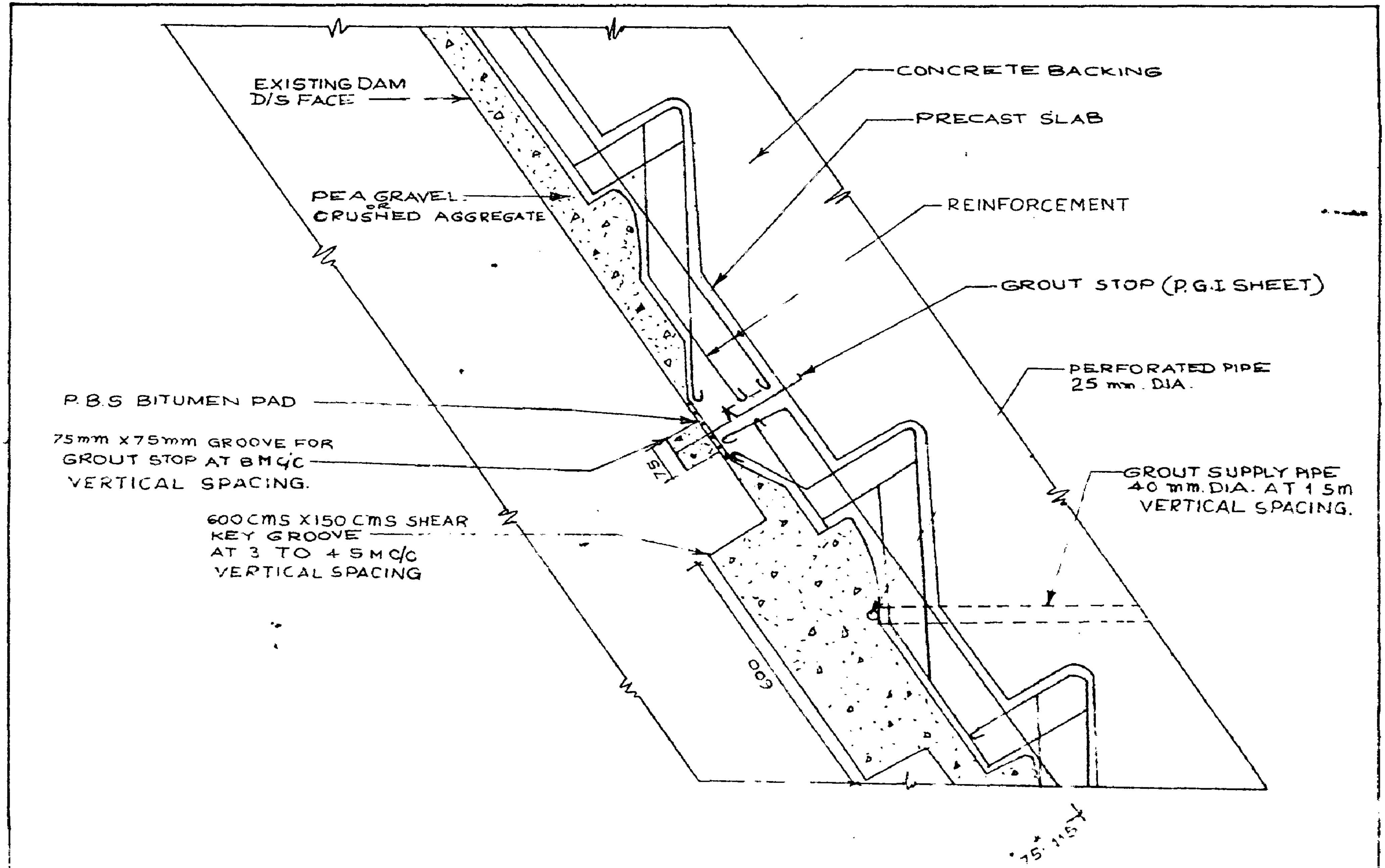
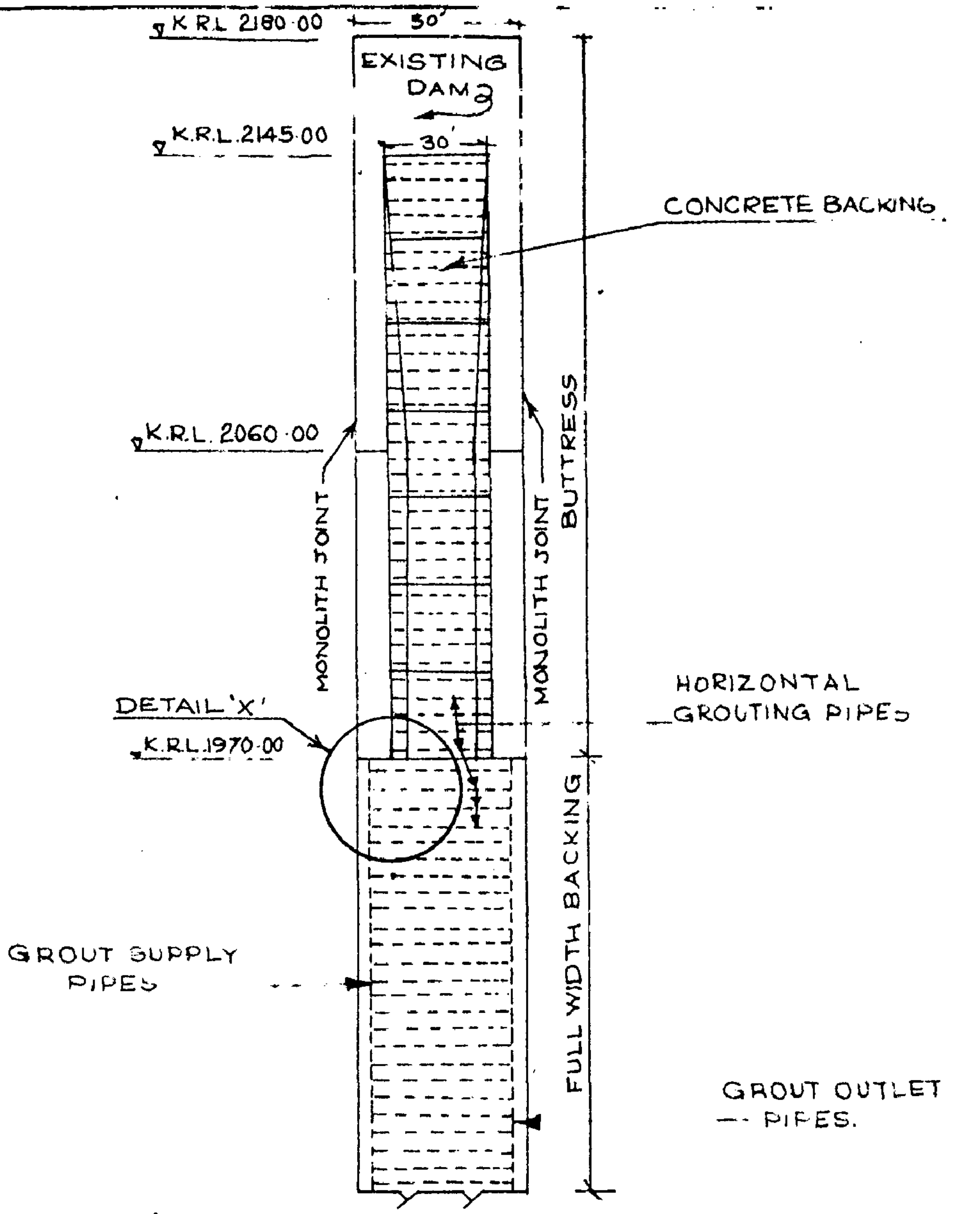
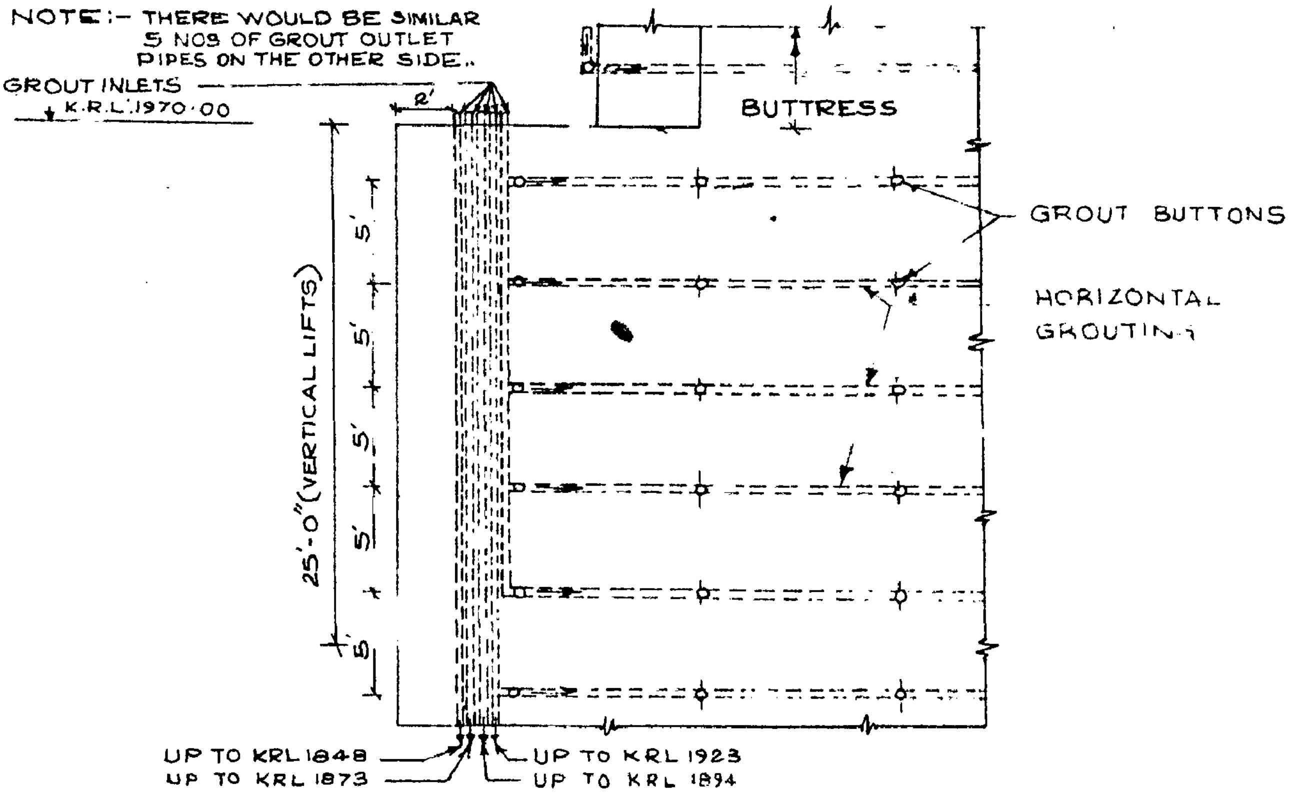



FIG. 4.1

DETAILS OF PRECAST SLAB SUPPORT FOR BACKING



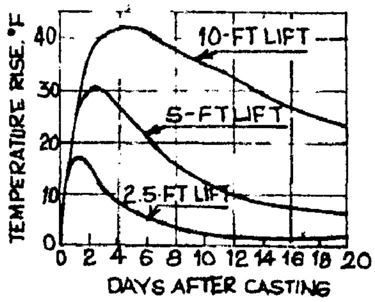

D/S VIEW OF BACKING OF KOYNA DAM, AGAINST ONE MONOLITH.
SCALE 1" = 60'



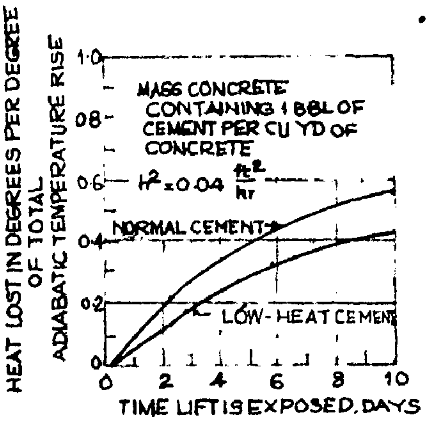

DETAIL 'X'
SCALE 1" = 10'

FIG 4.2

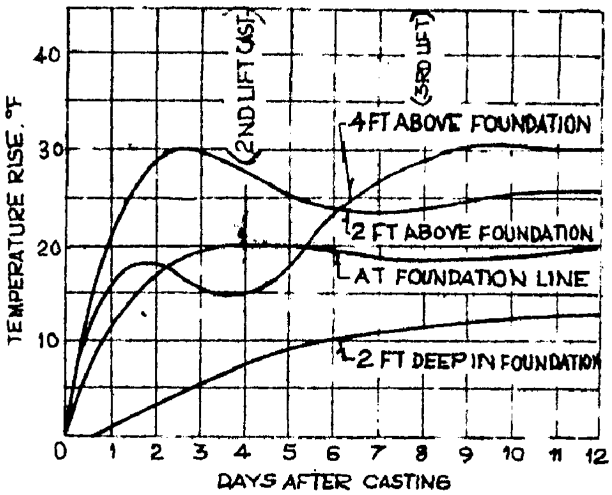
GROUTING ARRANGEMENTS FOR GROUTING OF CONTACT JOINT.



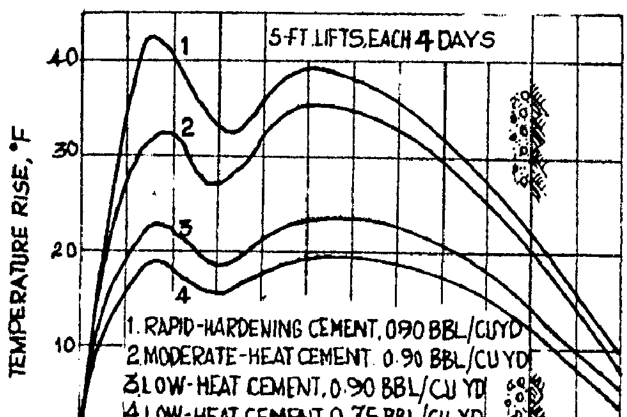

VARIATIONS IN TEMP. WITH TIME IN THE CENTRES OF UNCOVERED LIFTS OF 10,5,2 5 - FT. LIFTS




LOSS OF HEAT FROM THE TOP SUNFACE OF A 5 FT. LIFT




VARIATION IN TEMP WITH TIME IN CONCRETE CAST AT A RATE OF 5FT EVERY 4 DAYS




TEMP. DISTRIBUTION AT END OF 14th DAY SHOWING EFFECT OF TYPE & AMOUNT OF CEMENT IN CONCRETE

FIG. NO. 4·3

HEAT DEVELOPMENT IN RELATION WITH LIFT THICKNESS

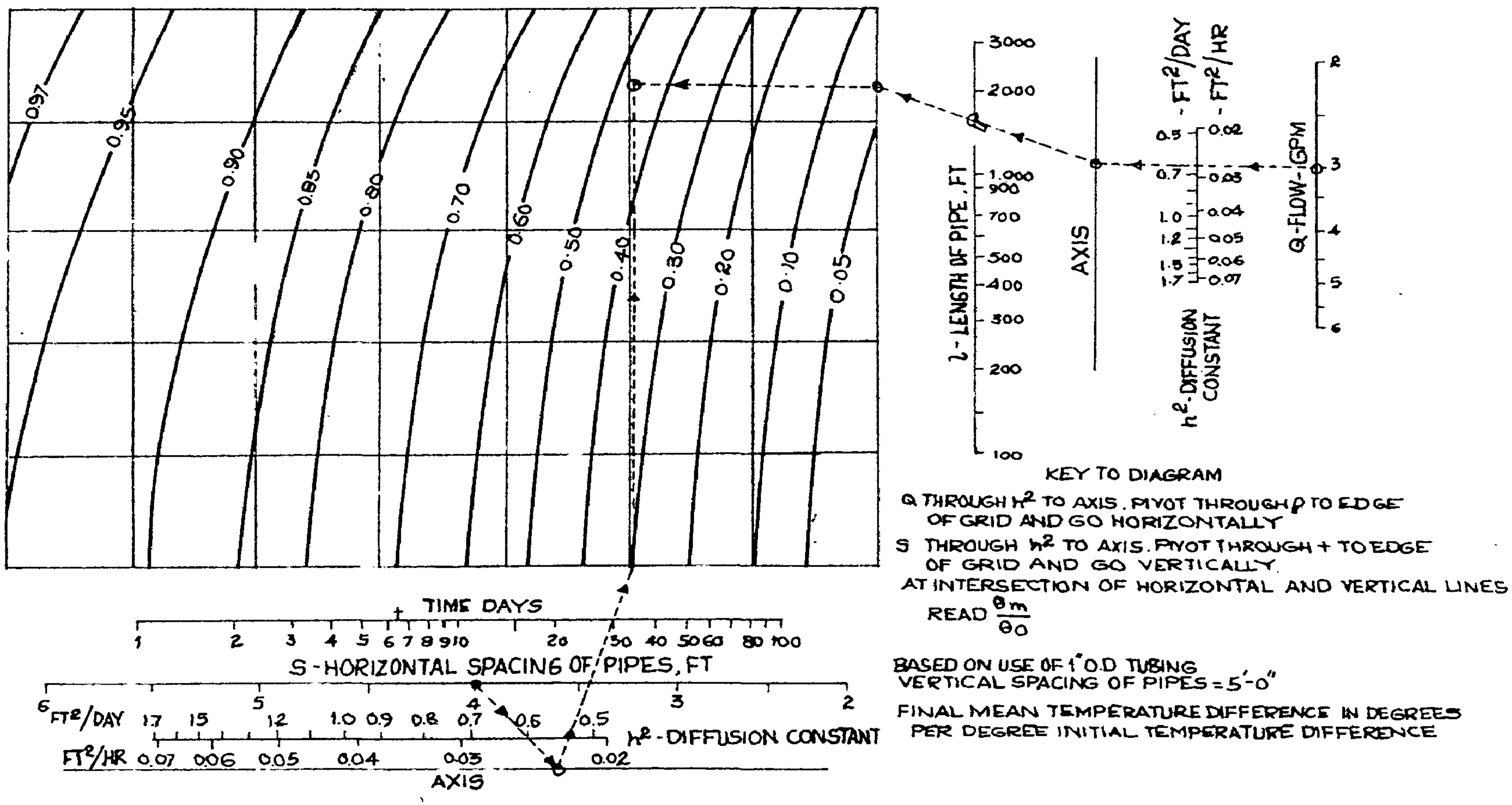



FIG. 4 4

MEAN TEMP. OF CONCRETE CONTAINIG COOLING COILS

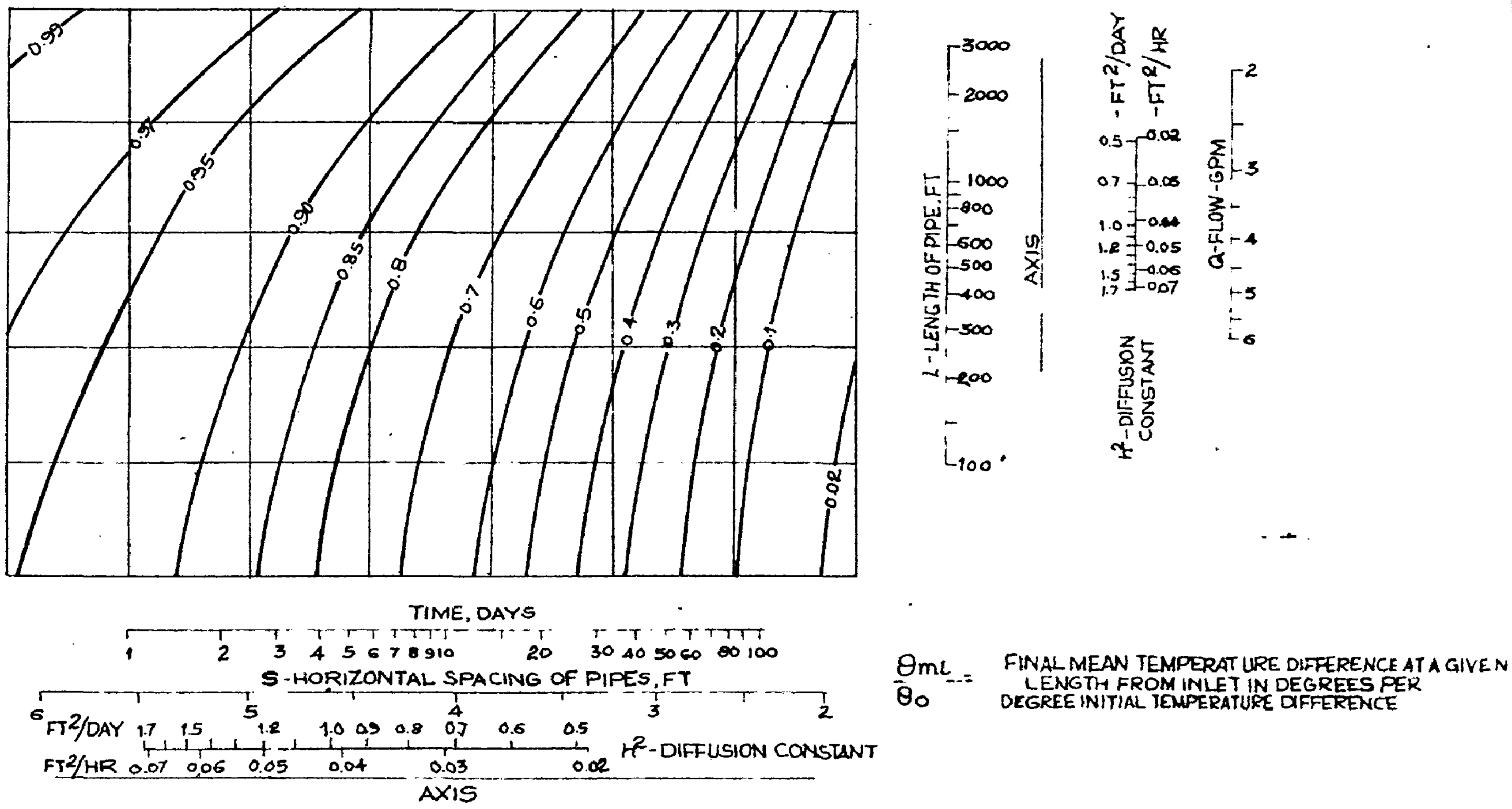



FIG. 4.5

MEAN TEMP. OF CONCRETE AT A GIVEN LENGTH FROM INLET OF COOLING COILS

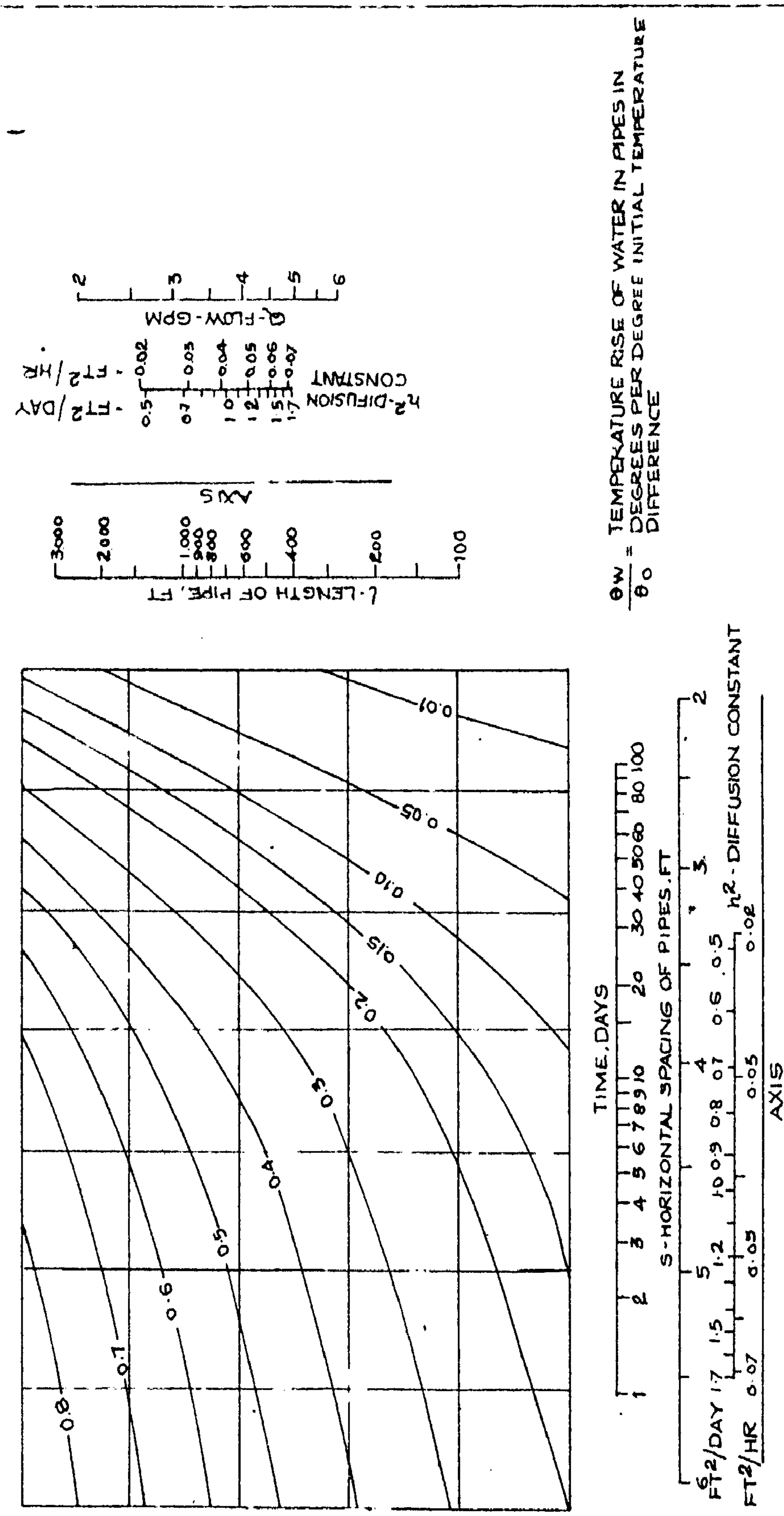



FIG. 4 6

TEMPERATURE RISE OF WATER IN COOLING COILS

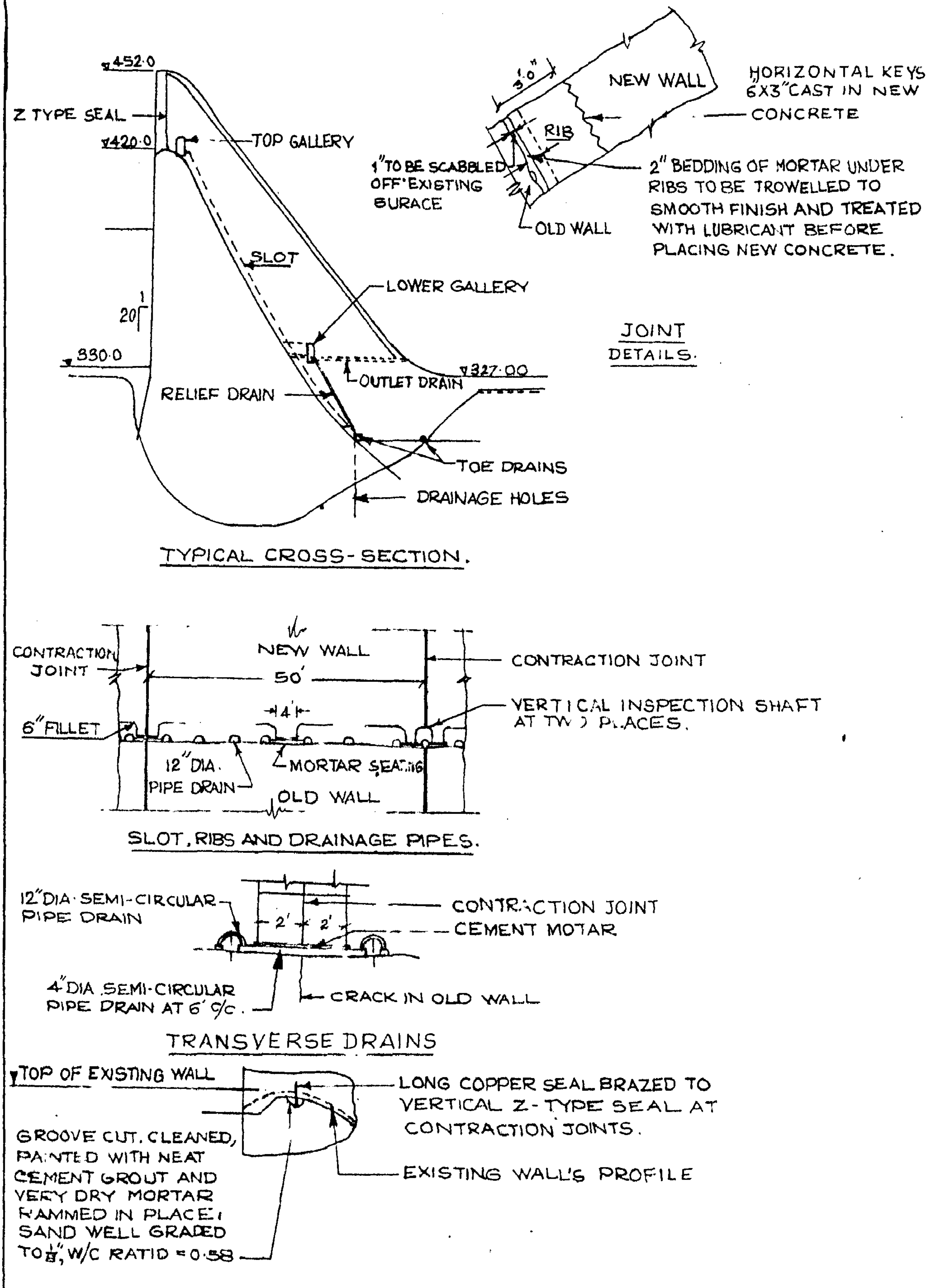
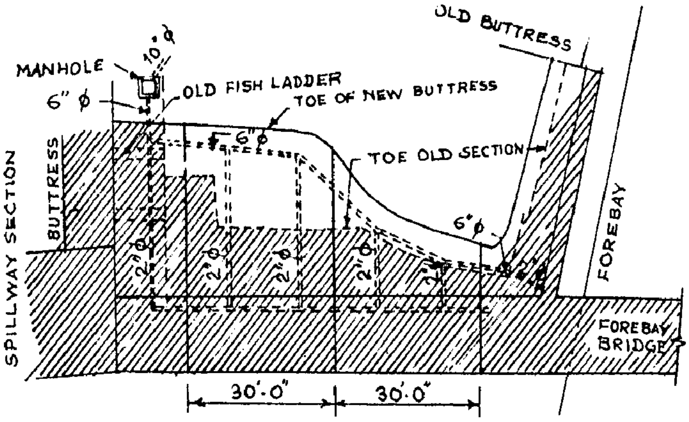


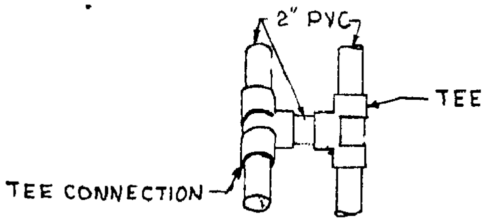

FIG. 4·17

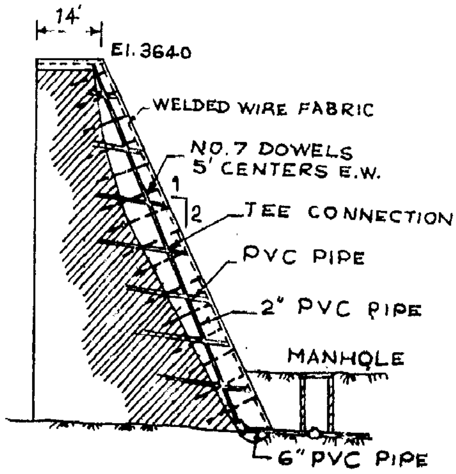
DETAILS OF STRENGTHENING OF MUNDARING WEIR, AUSTRALIA.




PLAN

0 10 20 Feet
SCALE




TYPICAL SECTION

FIG. 4·8

DRAINAGE LAYOUT IN RESPECT OF STRENGTHENING OF A GRAVITY DAM OF HAUSER DEVELOPMENT

# CHAPTER V

# DETAILED STUDY OF STRENGTHENING OF KOYNA DAM IN MAHARASHTRA STATE

## CHAPTER V

## Detailed study of strengthening of Koyna Dam in Maharashtra State

### 5.1 General

5.1.1 Koyna dam, the first rubble concrete dam constructed in India, is a gravity structure built across the Koyna river in the State of Maharashtra at an elevation of 580 metres (2000 ft) above the sea level. It is at a distance of 190 kms. South-South East of Bombay and is situated on the eastern side of the continental divide which lies within 60 kms of the west coast of India.

5.1.2 The Deccan Plateu on which the Koyna dam is situated had been held all along as an aseismic Zone. However the seismic activity which assumed sudden & unexpectedly serious proportions in the year 1967 proved otherwise. A severe earthquake of magnitude 6.5-7 on Richter Scale shook the region on 11th Dec. 1967. It caused wide-spread destruction in Koyna nagar Colony and its Surroundings, and also damaged the Koyna dam. Its epicentre was at 3 kms downstream of the dam, and its effect was felt upto a radius of 600 kms from the epicentre.

5.1.3 A committee of Indian experts consisting of engineers, geologists and seismologists was appointed by the Government of India for investigation of the causes of this seismic activity. UNESCO experts were also associated in this study.

5.1.4 From various observations, analysis of available data and different superficial expressions in this region, it has been concluded that, the tremors were due to the presence of an almost vertical deep seated fault in the North-South direction but concealed under the pile of basaltic layers. The committee of experts, ruled out the possibility held by some that, the earthquake might have been caused due to imbalance caused from percolation of water in the reservoir created behind the dam into the rock below.

5.2 Damage caused by earthquake of 11th Dec.1967:

Koyna dam is a gravity structure sub-divided into 53 monoliths out of which central seven monoliths form the overflow portion. Each monolith is 15 metres wide. The levels as mentioned in the discussion are in feet units.

The damage caused by this earthquake to the dam proper can be categorised as follows :-

i) Damage to appurtenant structures projecting from the top of the dam.

ii) Visible structural cracks on the face & inside of the dam.

iii) Local spalling of concrete.

5.3 The cross section of the non-overflow portion may be called as 'non standard'. It was originally intended to build a thinner section of the dam for a lower storage level @ KRL 2135, and to raise the height later by adding concrete on the downstream side. During construction, when the dam had actually been raised to some height in the first stage, it was decided to raise the height in a continuous operation to the ultimate storage level at KRL 2158.50' without recourse to its thickening on the downstream as far as possible. This was effected by building a section much wider than usual above levels KRL 2060 and 2065 by keeping the downstream slope very steep above these levels in the non-overflow monoliths. Also comparatively extra free-board was provided.

In this fashion, the technical difficulties in thickening the dam section on the downstream side were avoided by adding the necessary extra weight in the top portion of the dam section by making it much thicker than usual above KRL 2060. The comparison of the two sections i.e. section if construction were attempted in two stages and the section actually adopted for continuous construction in one stroke upto the final top of the dam, is given on figure 5.2. The spillway section was built with an overhang on the upstream side. Fig.5.2 shows the earlier planned section as well as the modified section.

5.4 The reasons for prominent cracking near KRL 2060 and for possibility of cracking at other elevations were investigated by theoretical analysis, dynamic photoelastic tests and fracture tests on plaster of paris models. These studies showed that, during the earthquake, large tensile stresses developed near the neck of high monoliths at KRL 2060 giving rise to cracks in this region which later extended due to repeated violent vibrations of the upper parts of these monoliths above the neck. It is possible that cracks may have penetrated deep into the body of the dam. The dynamic stresses induced on either faces of one of the high monoliths as estimated are shown in figure no.5.2.

5.5 The section of the dam was designed to withstand horizontal force due to earthquake corresponding to 5% of acceleration due to gravity under lake full condition, small tension being considered allowable. Under other loading conditions no tension was allowed. However during the earthquake of 11th Dec.1967, the dam was subjected to an earthquake force equivalent to ten times of that allowed for in the above design. The analysis was done on the basis of accelerograph record obtained from the instrument located in monolith 1-A at the time of the earthquake.

## 5.6 Immediate repair works

5.6.1 Considering the importance of the structure and the possibility of occurrence of earthquake shocks of the same severity as that of the ones already experienced and the damage already caused, the dam necessitated immediate repairs. However, as the permanent repair work would require a couple of years for its execution, some temporary but quick repairs were considered necessary for strengthening the structure to withstand shocks which may occur until the permanent strengthening works were completed.

Taking into account the possibility of recurrence of an earthquake of magnitude & intensity the same as that of 11th December 1967, which was the largest earthquake recorded in the area with epicentre close to the dam, it was considered necessary to design the measures for permanent strengthening capable of resisting the forces occuring during an earthquake of the same magnitude.

5.6.2 The temporary measures undertaken for repairs were the following :

1) Epoxy Grouting:

Epoxy grouting of cracks on either faces of the affected monoliths was resorted to. This helped sealing of cracks on the upstream and the downstream faces of the dam upto a depth of 3 metres or so. Entry of water through these into the body of the dam was thus prevented. This epoxy grouting which was done with all necessary care, helped also to bind effectively the concretes on either side of the cracks together in zones of the dam where stresses of high magnitude occur during an earthquake. Thus it served to make these critical zones as strong as they were before formation of the cracks. The strength of hardened epoxy and its bond with the concrete were tested by testing actual cores obtained by drilling across the epoxy filled cracks.

Before using epoxy resigns for grouting the cracks, extensive experimental investigation is necessary to determine the properties of xx various combinations of epoxies, hardners and diluents such as setting time, viscosity, bond strength etc. Wet conditions in the region of grouting can result in drastic reduction in the mechanical properties of the epoxies and their setting times. The selection of epoxy resin and hardner as well as their combination depends upon the condition of the cracks to be grouted i.e. whether wet or dry. It has been observed that epoxies can not penetrate very fine cracks for which other materials have to be used.

As an additional precaution to prevent entry of water into the body of the dam, the grouted cracks on the upstream face were covered sufficient margin above and below by gunite lining. This also took care of fine cracks which might not have been visible to the naked eye but present in the vicinity of noticeable larger cracks which were grouted. So also the possibility of imperfect grouting was taken care of.

The grouting operation was a difficult one and involved a race against time. It was done from catwalks erected cantilevering from the face of the dam. Barges were used to carry materials and equipment while grouting on the upstream face.

ii) <u>Prestressed Cabling</u>

This was another measure which was undertaken to fasten the portions of the dam above and below the prominent level of cracking i.e. KRL 2060. This operation was thus intended to increase compression in the zone of cracks and thereby supplement the improved mechanised bending achieved through epoxy-grouting of the cracks in this zone . In addition, fine cracks which may be present but which may have gone undetected and ungrouted would also close due to this compression.

Anchoring of the cables was done in the body of the dam itself. Anchoring region of the cables was adequately staggered to avoid concentration of stresses in the limited region of the dam. The lower ends of the cables were anchored in the zone of the dam which has compressive stresses even under earthquake condition. For tentative repair work, equivalent static seismic coefficients of Kh = 0.2 and Kv = 0.1 were considered for stress analysis. At KRL 2060, tensile stress of 32 t/m2 was expected on the downstream face with lake level at KRL 2000 and earth quake from downstream to upstream, while tensile stress of 42 t/m2 was expected on the upstream face with lake level at KRL 2135 and earthquake force acting from upstream to downstream. The prestressing done was expected to produce compressive stresses of 20 t/m2 and 1t/m2 on the upstream and the downstream face respectively at KRL 2060. Thus prestressing was effective in reducing tensile stresses especially on the upstream face during earthquake.

The cables were made out of 8 mm diameter wires, 64 in number. Load per cable was 270 tons. 11.44 cms. diameter holes were drilled to house these cables. 10 cables were provided per monolith.

5.6.3 Other measures immediately undertaken to improve stability were as below :-

i) Drainage holes of 75 mm diameter were drilled from top of dam to the top of operation gallery and from floor of operation gallery to the top of foundation gallery to relieve uplift pressure inside the dam.

ii) The drainage holes provided in the foundation of the dam from the foundation gallery were cleaned by brushes and ~~fxx~~ flushed with water to enable them to act efficiently in the relief of uplift pressures.

5.7 Permanent Strengthening Measures:

5.7.1 As already mentioned, Koyna Dam is of the straight gravity type and has been built of rubble concrete. The spillway is located in the central river portion and comprises monoliths 19 to 23 in full and monoliths 18 and 24 in part. The spillway is gated with six radial gates of size 41'x25'. A stilling basin on the downstream of the spillway provides for energy dissipation arrangement. Two penstocks have been provided, one each in monoliths 13 and 14 for supply of water to a power house proposed to be constructed on the downstream side. There are two galleries provided in the body of the dam which are approachable from outside as well as from electrically operated lift in M-18.

5.7.2 The Deccan traps provide a massive and a strong foundation for this structure, which is 103 m (338') high above the deepest foundation level (KRL 1842). The basalts in the foundation have a compressive strength of 1000-1500 kg/cm$^2$.

The foundation of the dam in the river portion was required to be taken down to keep it on massive basalt below 10 metres thick weak formation consisting of tuff breccia. Thus monoliths 15 to 23 have their foundation at KRL 1842. A shear zone cuts across the foundation of the dam at 60$^\circ$ to its axis in plan in monoliths 12,13 & 14.

The foundation rises up fast to higher levels as one proceeds from the centre towards the abutments.

After the catastrophic earthquake, core drilling done from the foundation gallery showed that, the contact between the foundation rock and the concrete of the dam was undisturbed.

5.7.3 Alternative Strengthening Measures:

5.7.3.1 The various alternatives considered for permanent strengthening of the non-overflow portion of this dam were the following -

i) Rock fill or earth fill backing,

ii) Full mass monolithic backing,

iii) Full buttress monolithic backing,

& iv) Partly solid mass and partly buttress backing.

5.7.3.2 The first alternative was not preferred, as it was felt that, the earth pressure could not be relied upon for effectively counteracting the inertia forces on the body of the dam due to phase difference between the two systems while oscillating during earthquakes. Theoretical analysis as well as model tests in case of proposed strengthening of Bhatgar dam in Maharashtra State carried out later also show that seperation occurs between the dam and the earth backing at higher elevations thus causing reduction in the stabilising earthpressure forces under earthquake conditions. This being thus doubtful case, the proposal was rejected.

Provision of full mass throughout the height for the purpose of strengthening was not favoured from considerations of dynamic behaviour of the dam-section during an earthquake. Dynamic analysis shows that, the acceleration response of a dam is highest at the top and decreases towards the base. It therefore follows that, placing a bigger mass at a higher elevation in the strengthening section will invite large overturning moment due to a long lever arm about the base of the dam, thus causing larger tensile stress on its upstream face. As the dam was to be strengthened mainly against the forces coming into play during an earthquake, this was an important consideration in the design of the strengthening section. Hence the alternative of provision of full mass throughout the height was not favoured.

The third alternative would have involved a buttress about 30 m wide at the base. This would have resulted in extra excavation, foundation treatment and shuttering costs, and that is why this proposal was not adopted.

The last proposal was considered most preferable as it struck balance between the second and the third alternatives. It involved provision of a solid mass at lower elevations thus thickening the whole monolith by about 21 metres with a buttress above it. The thinness of the buttress at higher elevations was advantageous from earthquake considerations. So also as the buttress had much greater exposed surface area per unit volume of concrete placed in comparison with the second proposal, cooling of concrete in the buttress was not expected to be necessary. Considering these advantages, this alternative was finally adopted.

5.8 Final Strengthening Proposal

5.8.1 The strengthened section was designed on the basis of design assumptions given later. The details of the strengthened section differ for the cracked deep monoliths and uncracked short monoliths. These are given in figure 5.1.

5.8.2 Out of 53 monoliths, 37 monoliths, namely 1-A to 18 and 24 to 41 were proposed to be strengthened. The remaining non-overflow monoliths near the abutments are nearly fully embedded in ground on either side and hence did not require any strengthening.

Dynamic analysis indicates that, the dynamic stresses in the overflow section are only about 60% of those developing in the non-overflow section at KRL 2060 & the base of the dam (40). In fact the dynamic stresses on the upstream face of the overflow section are less than those on the upstream face of the non-overflow section at all elevations. The reasons for lower stresses in the overflow section can be due to :-

(i) Lesser moments of the seismic force of inertia on account of lesser height of the overflow section.

& (ii) A comparatively much thinner section at the top where acceleration is the highest.

Thirdly there is no abrupt discontinuity in the profile of the overflow section as exists in the profile of the non-overflow section at KRL 2060. Such a discontinuity can cause high concentration of stresses which can give rise to cracking. This must have been one of the main reasons for cracking of the non-overflow section near KRL 2060.

5.8.3 The strengthening involved addition of a certain thickness of mass concrete from the foundation level of the existing section upto KRL 1960 in case of deep monoliths i.e. 9 to 18 and 24 to 31 and upto KRL 2000 in other monoliths against their entire width. From these levels upto KRL 2165, buttresses of about 9 m width were constructed in the centre of the monoliths to provide them support. The downstream slopes of the buttresses were made continuations of the downstream slopes of the thick sections below KRLs 2000 or 1960. The thickness of the addition was about 21 m in deep monoliths and about 7.5 m in short monoliths. The dynamic stresses that would develop in the event of an earthquake of the same magnitude as the one on 11th Dec.1967 are shown (50) in figure 5.3.

5.8.4 The buttresses have a trapezoidal cross section in plan. Wherever there is an appreciable difference in the slopes of the downstream face of the existing section and the downstream face of the buttress,their width on the downstream face has been varied for keeping a constant flaring angle for the side faces when seen in plan. Please refer figure 5.1.

### 5.8.5 Design Assumptions

The existing dam had been tested for a design seismic coefficient of five percent of acceleration due to gravity in the horizontal direction. However during the tremors which rocked Koyna dam on several occassions, the acceleration imparted was much more than that assumed as could be seen from accelerograph records. On the basis of earthquake record obtained from seismic instruments located inside the dam, the revised design was based on the following assumptions.

5.8.5.1 For the design of the strengthening section, an attempt has been made to relate the value of the seismic coefficient to be adopted in the estimation of stresses with the criterion of allowable stress condition in the dam. Thus a distinction has been made between the design seismic coefficient & dynamic seismic coefficient. The former is to be based on the factors such as seismicity of the region, coefficient adopted in the case of dams already constructed in the known seismic areas of India & other Countries which have not shown any signs of distress, nearness of the site with respect to faults etc. Development of tensile stress is not permissible when this coefficient is used in estimation of the stresses in the dam. On these considerations, ~~xx~~ a value of 0.12 was decided for the horizontal design seismic coefficient.

Dynamic seismic coefficient is the one which when used in the conventional method for the estimation of the stresses under the earthquake codition, gives stresses comparable with those estimated by the dynamic analysis carried out for the structure. If stresses are thus evaluated, the dynamic strength of material will have to be taken into account. In case of Koyna dam, such an analysis in case of the earthquake of 11.12. '67 indicated that a horizontal dynamic seismic coefficient of 0.5 gives a stress condition similar to the one obtained by carrying out the dynamic analysis for the observed ground motion.

On the basis of these considerations, the following criteria were evolved for the design :

I.

(1) Uncracked Monoliths :

Design seismic coefficient 'Kh' = 0.12 upstream to downstream for reservoir full condition.

Kh = 0.06, downstream to upstream for reservoir water level at KRL 2000'.

No tension to be permitted.

(2) Cracked monoliths:

Kh = 0.20, upstream to downstream, lake full.

Kh = 0.10, downstream to upstream, lake level at KRL 2000'

No tension to be permitted.

These loading conditions, we may term as the design conditions

The higher design seismic coefficients for the cracked monoliths are to allow for the fact that, the concrete in cracked monoliths is not considered able to develop the same ultimate tensile strength in case of emergencies as can the concrete of uncracked monolit

II.

Estimation of dynamic stresses also to be done using the dynamic seismic coefficient.

| | |
|---|---|
| Dynamic seismic coefficient : | 0.50 uniform from the top of the dam upto KRL 2145'. The same to be linearly reduced to 0.10 at KRL 1845'. |
| (for inertia forces) | |

To calculate the increased forces due to water pressure during the earthquake, the same values for 'Kh' as are given in 'I' above for cracked and uncracked monoliths to be used.

The stress calculated on this basis should not exceed a value of 30 kgs/$cm^2$ - i.e. tensile strength of concrete

This condition of loading has been termed as 'Checking Condition' or 'Emergency Condition'.

5-8.5.2 As regards uplift pressures, for horizontal sections below the level of the bottom of the buttress, full uplift is assumed at the heel, reducing to 1/3 rd its value at the line of drains, further reducing to zero at the downstream toe of the new backing. For horizontal sections at and above the level of the bottom of the buttress, the uplift pressures are assumed decreasing linearly from whatever value at the drains to zero at the toe of the existing section.

5.8.5.3 Modulus of elasticity has been assumed the same in the old and the new concrete. Also modulus of elasticity of rock under the foundation of the new backing has been assumed the same as the modulus of elasticity of rock below the existing structure

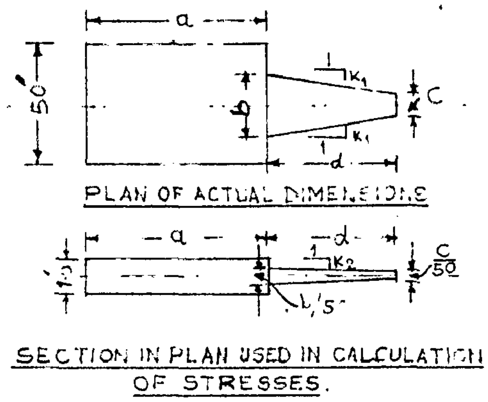
Gravity analysis has been used in calculation of stresses.

5.8.6 <u>Some salient features of stress estimation</u>:

These can be mentioned as under :

i) While calculating the stresses in the dam, the usual method of taking cantilever of unit thickness could not be used. This was so because of the buttress addition. Thus the section in plan @ any elevation was obtained by taking one foot thickness of the existing dam along with a buttress section having its dimensions (parallel to the length of the dam) reduced to 1/50th of its actual dimensions. The dimension of the buttress perpendicular to the length of the dam was kept unchanged as shown below. This is justified as the moments o

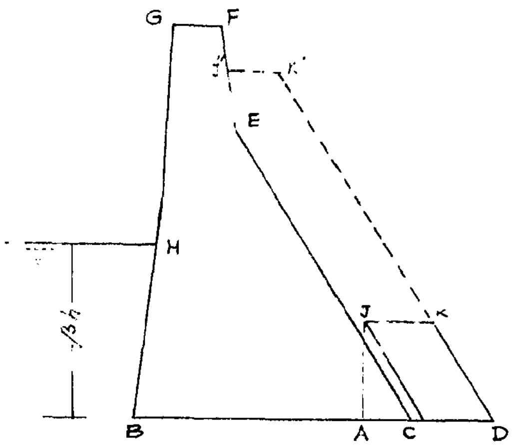
~~to self~~ weight and the section modulus, which govern the stresses are directly proportional to the thickness of the slice being considered for stress analysis by the conventional method.




(ii) As would be evident from the previous chapter, the reservoir level at the time of laying gap concrete has an important effect on the stress distribution within the composite structure. The final stresses are obtained by super-position of locked up stresses and the stresses due to the incremental loads. In determination of locked-up stresses, some assumptions are involved. The stress picture is also affected by the method of construction. The manner in which these stresses have been calculated in the present case is discussed below in detail.

The newbacking section when built separate from the existing section leans towards the upstream side, the slope of its upstream overhanging face being 0.725 H:1v in cracked monoliths. Therefore if built in such unsupported condition to unrestricted heights, tension would have developed on the downstream face. The maximum unsupported height without causing tension on the downstream face was calculated as 9 metres at lower elevations. It was therefore necessary that by the time the new section was raised by 9m, its bottom most lift should fully shrink so as to enable placement of corresponding lift in the gap portion. Only in this fashion, the work of concreting in a monolith can be kept a continuous operation.

When such a plan of concreting is followed we can find the state of stress x say @ the foundation level by considering the forces acting corresponding to lake water level just prior to the placement of bottom-most lift of gap concrete. While calculating these stresses, question arises as to what portion of the weight of the backing concrete affects. the stress distribution in the width of the old section. This is a matter of judgement as there is no direct method of such evaulation. The calculations made in this case were as follows:




a) The stress distribution on base BC is found under the action of water pressure (depth $\beta h$ ), weight of the existing section GHBCEFG and uplift forces.

b) The stress distribution on base AD is found due to weight of the new backing section JCDK. JCDK is the mass which can be seperately constructed as shown without developing objectionable tensile stresses at the toe 'D'.

c) The stress diagram obtained in the above step (b) for the portion AC is considered as a load. The stress distribution on base BC is found due to this load.

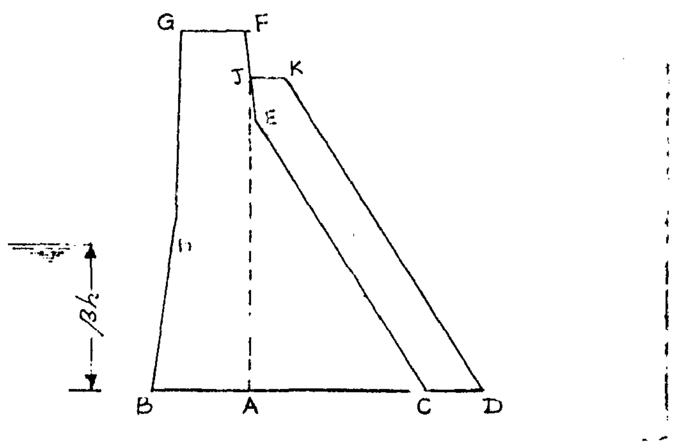
d) The stress distributions on base BC obtained in steps (a) & (c) are superimposed to find locked up stresses. The stresses on the part CD remain the same as calculated in step (b).

e) The gap concrete in the bottom most lift is now assumed to be placed. Also the backing for the full height is considered to have been completed, including the gap filling. Water level is raised to the final level. Under these conditions and with earthquake forces acting, assuming monolithic action, the stresses due to loads in excess of those considered in the earlier steps are calculated taking BD as one base.

f) The stress diagrams obtained on base BD in steps (d) & (e) are super-imposed to get the final stress picture at the level BD.

Stresses at any other level can be calculated in a similar manner.

This method of calculating stresses was however not actually used as it was thought of at a later date. The calculations done earlier were on the following lines.




a) Stress distribution on base AD due to weight of section JECDK was found.

b) The stress diagram obtained in the above step for the portion AC was considered as a load. Stress distribution on base BC was found due to this load.

c) Stress distribution on base BC due to water pressure (depth '$\rho$h'), weight of existing section GHBCEJFG and uplift forces was calculated.

d) The stress diagrams obtained in steps (b) and (c) were super-imposed to find locked up stresses. The stresses on part CD remained the same as calculated in step (a).

e) The stresses on the full base BD were found due to incremental loads comprising of increased water pressure on account of rise of water level upto F.R.L., increase in uplift & earthquake forces. Monolithic action was considered.

f) The stress diagrams obtained on base BD in steps (d) & (e) were super-imposed to get the final stress picture at the level BD.

This second procedure although not accurate was however found to give results quite close to those which were obtained by the first method and hence due to simplicity was adopted.

These methods of stress calculations thus involve an assumption about the effect of the added mass on stresses in the old dam portion. Considering the method of construction adopted, in which anchor bars of mild steel have been provided on the downstream face of the existing dam, transfer of part of the weight of the seperately constructed backing to the existing section might be actually taking place through these anchor bars also. The provision of these anchors was however not made on such considerations.

(iii) The stresses on the upstream face of the dam were of interest, as uncontrolled joining of the two concretes irrespective of the reservoir water level could cause undesirable tensile stresses on the upstream face. Therefore for different reservoir elevations, stresses on the upstream face of the dam were calculated at several elevations. From these computations, curves showing stresses on the upstream face at all elevations corresponding to different reservoir water levels at the time of bonding were plotted as shown in figure No.5.3.

From these curves it is evident that, the gap concrete can be placed at any reservoir elevation below K R. 2100, without causing tensile stresses on the upstream face at any elevation. It is also seen that, as we proceed from the foundation level upwards, joining concrete can be placed with reservoir water standing at successively higher levels without causing tensile stress on the upstream face.

A table giving the maximum water levels permissible at the time of joining of the new mass with the main dam could be prepared from these curves. Such a table was a useful guide in framing a continuous construction schedule, as well as during construction control.

5.8.7 Type of Contact Joint between the existing section and the added mass

5.8.7.1 As already seen in chapter III, the joint between the two concretes must be strong enough to be relied upon for transfer of stresses from the old to the new section as in the case of a monolithic mass.

Owing to the setting up of undesirably high shrinkage stresses at the joint as well as tensile stresses on the upstream face when the new concrete is placed directly on the existing dam, the proposal of placing new concrete directly on the existing section was not adopted.

5.8.7.2 Two alternatives were considered for forming this joint. The first consisted of keeping ~~at~~ a 7.5 cms to 15 cms wide gap packed with peagravel of size 9 mm to 19 mm under precast concrete panels supporting the new concrete and grouting the gravel under appropriate temperature conditions in the new mass and under appropriate reservoir water level. However laboratory and field experiments conducted to determine the shear strength of the grouted joint did not give encouraging results. Therefore this proposal was not favoured.

5.8.7.3 Second alternative of leaving a sufficiently wide gap between the existing section and the new backing initially and filling it with concrete when the new concrete completed its shrinkage after adequate cooling was considered as a better method from all aspects. This led to the provision of a 1.2 metre ~~wd~~ wide (horizontally) gap between the two concretes, to be filled with precooled concrete subsequently. This method had also the advantage of laying concrete of nearly the same characteristics against the concrete of the existing structure.

5.8.8 Design of joint :

The design of the contact joint presented problems. Due to the large height of the dam and the earthquake forces to which it was likely to be subjected, shearing and tensile stresses of appreciable magnitude were expected to occur in the region of the joint. Therefore adequate provision to resist these stresses had to be made.

5.8.8.1 The resistance acting at the joint of the old and the new concrete due to various treatments consist of the following :

i) Resistance due to bond which consists of two parts -

a) Cohesion part of the bond between the two concrete surfaces obtained by roughening and cleaning the existing dam surface,

and b) Frictional resistance part of the bond between the two concrete surfaces which depends upon the normal stresses acting on the joint and the angle of internal friction.

ii) Shear resistance offered by shear key concrete provided on the downstream face of the dam as well as on the upstream face of the backing concrete.

& iii) Shear resistance provided by anchor bars embedded adequately in the old as well as the new concrete sections.

The resistance which can be offered by each of these can be estimated. However if the resistances offered by all of them are to be added up, it is necessary that their estimated strengths come into effect simultaneously. In other words, the stresses in all these devices should reach their permissible limits at the same time. That is to say, when the limit of 5.45 kg/cm$^2$ (80 p.s.i.) for cohesion is reached, the limit of 1700 kgs/cm$^2$ (25000 p.s.i.) in steel should also be reached. If such relationships could hold, all these resistances could be depended upon for acting together in unision, assuring at the same time a most economical design.

However the difference between the allowable shearing stresses in reinforcing steel and cohesion being too large, it is expected that, they will not reach their permissible limits simultaneously. Similar is the case when shear strength of concrete in keys is considered with reinforcing dowels as the modular ratio for shear between steel and concrete varies from 4 to 12.

5.8.8.2 From field shear tests conducted on blocks of new concrete cast on roughened surfaces of the existing dam it was found that, the cohesion part of the bond strength between the two surfaces of concrete had an average value of 10.5 kg/cm$^2$ (155 p.s.i.), the minimum value observed being 7.80 kg/cm$^2$ (111 p.s.i.) at the age of 28 days. The angle of internal friction was observed to be 55$^\circ$ on an average. These are the values based on 13 tests conducted in the field. Similar tests but with concrete test blocks cast on cleaned but unroughened surfaces of the existing concrete showed that, the cohesion part of bond strength of such a joint is about 5.65 kg/cm$^2$ (83 p.s.i.) on an average. From these tests it was concluded that, it was safe to assume cohesion part of bond to resist shear stresses upto 5.45 kg/cm$^2$ (80 p.s.i. In emergency condition, which is a rare condition, the permissible shear stresses which can be resisted by cohesion alone were limited to 7.5 kg/cm$^2$ (110 p.s.i.). It was also assumed that, the cohesion will reach a value of 17 kg/cm$^2$ (250 p.s.i.) at the age of 90 days, thus giving adequate factor of safety. The concrete used for making the block had a compressive strength (cylinder) of 150 kg/cm$^2$ at 90 days.

Results of tests on concrete blocks of size 0.6 m (width) x1 metre (length) x 0.4 metre provided with a shear key at the bottom of size 0.6 x 0.6 m indicated that, when cohesion on the area of contact (excluding the shear key area) is fully mobilised contribution of shear key to the shearing resistance is negligible.

5.8.8.3 From analysis of stresses it was found that, the unbalanced shear stress at the joint (i.e. shear stress minus product of normal stress and tangent of angle of internal friction) in case of short monoliths did not exceed 2.72 kg/cm$^2$ (40 p.s.i.) in design condition and 4.07 kg/cm2 (60 p.s.i.)in emergency condition. In case of deep cracked monoliths, the corresponding maximum values were 5.45 kg/cm2 (80 p.s.i.) and 7.5 kg/cm2 (110 p.s.i.) respectively. Thus from considerations of shear stresses alone there was no necessity of reinforcing the joint by shear keys and dowels. However the permissible values of shear stresses which can be resisted by cohesion alone as stated in the earlier para could not be relied upon whereever there was liklelihood of separation at the interface due to tensile stresses occuring under any of the two conditions. It was assumed that the interface joint can be expected to withstand a tensile stress of 1.36 kg/cm2 (20 p.s.i.) without seperation. Therefore, the shear stresses in the zones with normal stress more than 1.36 kg/cm$^2$ (tension) were provided for by anchor bars alone. Please refer to figure number 5.4 which indicates the distribution of shear and tensile stresses on the joint.

5.8.8.4. Design of Dowel bars

Where shear stresses and tensile stresses are to be resisted simultaneously by the dowels, as in the case of region between elevations 2040 & 2140 of deep monoliths where tensile stress exceeds 1.36kg/cm2 (20 p.s.i.), the yield shear stress in steel as given by the following relationship was adopted in the design.

Yield stress in shear = 0.557 x yield stress in Tension.

Some authors give the ratio of yield stress in shear and tension as 0.60 also.

When designing the reinforcement, 90% of yield stress was assumed allowable in 'emergency condition' and 60% of yield stress as allowable in 'design condition'. As use of mild steel bars would have resulted in too close a spacing as to be impracticable, it was decided to use cold twisted deformed bars having a minimum yield strength of 4080 kg/cm$^2$ (60,000 p.s.i.) and an ultimate strength of 4760 kg/cm2 (70,000 p.s.i.) in tension.

The following relationship is necessary to be satisfied when the steel is provided to resist the shear as well as tensile stresses.

$$\left(\frac{fs}{ps}\right)^2 + \left(\frac{ft}{pt}\right)^2 \leqslant 1$$

where, fs - actual shear stress in steel,
ps - allowable shear stress in steel,
ft - actual tensile stress in steel,
and pt - allowable tensile stress in steel.

On the basis of the above considerations, the requirement of dowel bars for resisting shear stresses and tensile stresses was calculated.

5.8.8.5 The interface treatment in case of the joint between the two concretes was thus provided as below :

i) Shear keys of 30 cms width and 22.5 cms depth at 3 metres c/c in all regions.

ii) Roughening of the downstream concrete surface of the existing dam and the upstream surface of new backing concrete in all regions.

iii) 32 mm diameter cold twisted deformed bars (IS 1786-1968) at 1.5 metres centres bothways in all regions where tension is less than 1.36 kg/cm2 (20 p.s.i.) i.e. in all region of uncracked monoliths and between KRLs 1842 and 2040 in case of cracked monoliths.

iv) Provision of steel of the same quality as above as anchors in regions where tension exceeded 1.36 kg/cm2 (20 p.s.i) as per the following table. (i;e. between KRLs 2040 and 2140 in cracked monoliths).

Table No. 6

| Region | Governing condition | Steel in square cms per square metre area of the joint. |
|---|---|---|
| Between KRLs 2040 & 2065 | Emergency | 36.30 |
| Between KRLs 2065 & 2140 | Emergency | 29.75 |

It may ne mentioned here that, there is thus likelihood of separation between the two concretes in the zone where tensile stresses exceed 1.36 kg/cm$^2$ (20 p.s.i.). However, as the separation does not prevent transfer of stresses across the joint as the dowels connecting the two concretes are designed for this purpose, monolithism is still ensured. The danger of entry of water in the ~~x~~ cracked portion of the joint is however not existent as any crack at the joint will not be connected with the reservoir water as in the case of heightening of dams.

5.8.8.6 Embeddment of reinforcement

Use of 32 mm diameter bars has been generally made for shear and tensile reinforcement. The cross sectional area of one bar is 8 cm$^2$ and therefore if the tensile stress in the zone in which the steel is provided is 3.13 kg/cm$^2$ (46 p.s.i.), the force in the bar will be,

$$3.13 \times \frac{8}{29.75} \times 100 \times 100 = 8400 \text{ kgs.}$$

$$\therefore \text{ Stress in bar} = \frac{8400}{8} = 1050 \text{ kg/cm}^2$$

Thus the stress in the bar is much less as compared to its strength. As the bar is in tension, it is necessary that it has sufficient anchorage depth in order that it does not get pulled out. As the bar has been anchored by being grouted in a hole drilled in the concrete of the old section of the dam, it was necessary to determine the length of this type of anchorage against pull out. This was necessary as the bar may be pulled out due to bond failure, which may take place either at the surface of contact between the bar and the grout or the peripheral contact of grout with the drilled hole. During tests it was always found that, the bond fails at the surface of the bar. In case of 20 mm diameter torsteel bars, which were used in pull out tests, it was found that, with cement grout having water cement ratio of 0.55 and proportion consisting of 1 part of cement to 4 parts of fine sand by volume, there was significant increase in the load required to pull out the bar as the depth of anchorage was increased, but that after about 1.2 metres depth, the increase was not substantial. With too large depths of anchorage, proper filling of the holes may also be difficult to be ensured and thus about 1.5 metres depth of embeddement may be considered as satisfactory. Actually, the tension in the bars is much below that required to pull them out and hence still lesser depth of anchorage can be considered to be adequate. However, 2.4 metres depth of anchorage in the existing dam and 2.025 metres depth inside the new backing concrete with 0.975 metre length in the gap was provided to get an adequate factor of safety.

5.8.8.7 Cost of Joint Treatment:

The total cost of the work of strengthening was estimated at about Rs.4.00 crores. Out of this, the cost of the joint treatment provided is of the order of Rs.20.00 lakhs. Thus the cost of joint treatment works out to 5% of the total cost in the case of Koyna Dam.

5.9 Foundation Widening :

5.9.1 It was decided to excavate in rock for resting foundation of the new backing at the same level as that of the existing structure at its downstream toe. As the distance of the blasting charge from the structure varied from 0 metre to 21 metres, experiments were conducted to determine the safe blasting charges for blasting rock at different distances from the structure.

5.9.2 Table no.4 of Chapter III ... as given by Mr. Langefors was used as a guide to assess the safe charge level for blasting. The maximum particle velocity was used as the damage criterion. The ratio between the experimentally observed ground particle velocity and corresponding Langefors particle velocity for the same charge level was utilised for computing safe charge for excavation. The experimentally observed ground particle velocities were 37% of the corresponding Langefors particle velocities on an average. In other words the rock at the site was found to be poorer in transmitting the energy of the blast in comparison to Swedish rocks.

5.9.3 On the basis of these considerations and taking into account the fact that the dam had been damaged during the earthquake, the total safe charge (Q) in kgs/delay (delay interval greater than or equal to half second) distributed in any number of holes at any distance (R) in metres (Not less than 6 metres) was used as per table given below :-

Table No.7

| Distance 'R' in metres | Charge (Q) in kg/delay (Delay interval 0.5 sec) |
|---|---|
| 6 | 0.60 |
| 9 | 1.00 |
| 12 | 1.70 |
| 15 | 2.30 |
| 18 | 3.00 |
| 20 | 3.60 |
| 25 | 5.00 |
| 30 | 6.60 |

The total safe charge (Q) in kgs/delay distributed in any number of holes at any distance (R) in metres (less than 6 metres) was specified as per another table given below. However, the use of charges on the basis of this table was made only when the rock between the location of the charges and the structure to be protected was presplitted to 2.4 metres depth for the complete length of the monolith.

Table No.8

| R in metres | Q kgs/delay |
|---|---|
| 1 | 0.04 |
| 2 | 0.11 |
| 3 | 0.20 |
| 4 | 0.32 |
| 5 | 0.45 |

When presplitting is not done, the permissible charges were as per table given below in case of distances less than 6 metres. These were also the charges to be used in presplitting operations.

Table No.9

| R | Q |
|---|---|
| 1 | 0.02 |
| 2 | 0.04 |
| 3 | 0.08 |
| 4 | 0.12 |
| 5 | 0.17 |

5.10 Design of mix :

5.10.1 Although the existing dam has been constructed of rubble concrete, mass concrete using 150 mm maximum size aggregate was used for the strengthening section. Although use of a similar construction material would have been desirable, its use would have required the erection of the special plant and equipment which would have not been possible within the time available, it being necessary to complete the strengthening work as quickly as possible.

5.10.2 So also with the use of rubble concrete, it would not have been possible to install embedded pipe cooling system if required to complete cooling of the new concrete within a period of one and a half months which was necessary for continuous concrete placement. Also concrete in the gap could not have been rubble concrete in any case.

5.10.3 Considering all these factors, it was decided to use 150 mm max. size aggregate mass concrete. For the gap, 75 mm max. size aggregate concrete was used due to difficult conditions of placement and to avoid seggregation.

5.10.4 The cement used was mainly from the Shahabad cement factory. Cement was being purchased from Bagalkot and Shahabad cement factories for the Koyna Project works. As shahabad cement had shown lesser generation of heat, it was specifically used for the strengthening work.

5.10.5 The slump specified for the mass concrete was 25 mm, while that for the gap concrete was 50 mm. Strength of 150 kgs/cm$^2$ was specified at the age of 90 days for the mass concrete. To satisfy requirements of slump and strength, cement contents of 177 kgs/m$^3$ and 197 kgs/m$^3$ were required in case of mass concrete and gap concrete respectively.

5.11 Important Construction Features and Aspects :

5.11.1 Controlled blasting :

As excavation for foundations for the backing concrete had to be done close to the existing dam, restriction on blasting charges had to be imposed for safety of the dam, as well as nearby appurtenant structures. The charges, which were decided to be safe for 'no crack condition', were used as given earlier. The work having been entrusted to contractors, utmost care had to be taken in exercising a check on the use of correct charges for blasting.

As mentioned in the next paragraph, dismantling of a portion of right bank masonry guide wall had become a matter of top priority in the working season of 1969-70. Earlier it was thought that, this work could be done by pneumatic breakers. However when actually attempted, it was seen that, this would have taken too long a period and hence, the masonry in the portion of the wall to be demolished was blasted using the same charges as given earlier in the beginning and with 50% extra charges at a later stage to expedite the work. Use of this extra charge was quite safe and did not damage the nearest portions of the guide wall which were to be kept in tact. The use of extra charge in blasting masonry was considered justifiable as the transmission of energy through masonry can not be as efficient as through rock and it proved to be so.

5.11.2 Problems in Ms 15 to 18

5.11.2.1 In monoliths 15 to 18, the excavation in rock had to be done through a depth of 16.5 metres. This was necessary as band of weak material existed between KRLS 1889 and 1852 which was not considered acceptable even in the construction of the original dam. Some difficulty was experienced in this strata as drill rods used to get jammed during drilling operations. Another difficult task was the removal of the backfill material in between the downstream face of the existing section and the original face of excavation. Removal of materials from the deep pit had to be done by skips which were x lowered and hoisted by cranes.

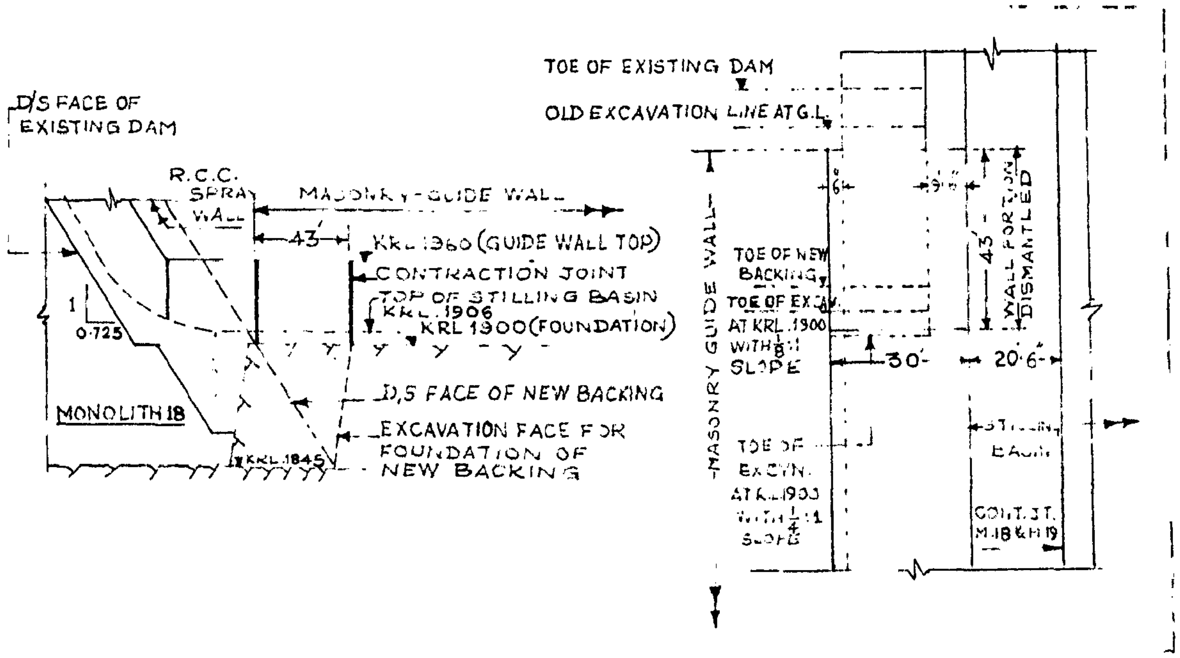
5.11.2.2 From the sketch of the right bank guide wall given later it will be seen that slope of 1/4 H:1v with berms at suitable intervals etc. could not be given to the downstream excavation face as otherwise dismantling of additional portion of masonry wall would have been involved. It was therefore decided to excavate at a slope of about 1/8 H:1v. Similarly to get enough width for backing concrete in M-18, excavation face in a direction at right angles to the length of the dam below the foundation ledge of right bank guide wall in reinforced concrete of 2.85 m base width had to be done at a slope of 1/10 H:1v. This was achieved by a method similar to that of line drilling.

5.11.2.3 Another difficulty in this portion was the fractured zone crossing monoliths 14 and 15. The width of this zone decreased at lower elevations and was about 1.2 metres at KRL 1845.00. The material in it consisted of gauge enclosing basaltic pieces. This material was excavated by manual labour using pavement breakers where essential, but mainly by pick-axe to a depth of 3 metres below the general foundation level. This trench was concreted, keeping therein pipes for drilling grout holes and drainage holes at a latet date. Also reinforcement was placed spanning across this zone in the foundation as well as for some length along the downstream excavation face for load transfer to the rock on either side of the weak zone.

5.11.3 Right Bank Guide Wall

5.11.3.1 The general top of rock level in Ms 15 to 18 was at KRL 1900 or so and the excavation for foundations was done here upto KRL 1845.

A peculiar condition arose in the provision of backing concrete in monolith 18 on which rests the right bank training wall. Part of this training wall is in reinforced concrete and part in masonry. The position is as shown in the figures given below :




5.11.3.2 The thickness of the R.C.C. wall is 2.85 metres only, while that of masonry wall is 9 metres. It will be seen that, it was necessary to dismantle 12.9 m length of masonry wall between contraction joints to enable provision of backing concrete in M-18 (its non-overflow- portion).

5.11.3.3 As mentioned earlier, dismantling of this portion was done by controlled blasting using somewhat more blasting charges as compared to those used in blasting rock as there were contraction joints on either side of the portion to be demolished and as more absorption of energy of the blast was expected to take place in masonry than in rock.

5.11.3.4 This dismantling by blasting presented no difficulty. Earlier it was thought that, the dismantled wall should be replaced by a similar masonry wall. However late receipt of the imported batching and mixing plant resulted in delaying the concreting programme in monolith 18, which in turn affected the above decision taken earlier. The situation was reviewed in Jan.'70. Considering the necessity of closing the gap created by dismantling part of the training wall of the stilling basin before the onset of monsoon, it was decided to close the gap with R.C.C. wall having its horizontal leg on the stilling basin side. Thus the construction of the wall could be made independent of the programme of concreting in monolith 18 and hence was favoured.

5.11.3.5 This R.C.C. wall, proposed as a substitute for the dismantled masonry wall, could have been anchored with stilling basin concrete at its bottom; however, the effectiveness and reliability of such an anchorage could not be compared with the safety and reliability obtained with a monolithic construction. From this point of view, it was decided to cut out stilling basin apron concrete in front of the 12.9 m long gap upto transverse contraction joints in the stilling basin concrete as shown in the figure drawn earlier. To achieve proper and smooth cutting, a closely spaced line of jack hammer holes was drilled along the periphery of the concrete portion to be cut out. This line of holes also served to prevent the transmission of energy of the blast to the adjacent intact concrete portions of the stilling basin. Controlled blasting was done to remove bulk of the apron concrete, some portion around the periphery being cut out by pavement breakers. This operation having been completed, concreting of the R.C.C. wall could be started and finished before the onset of monsoon in the year 1970.

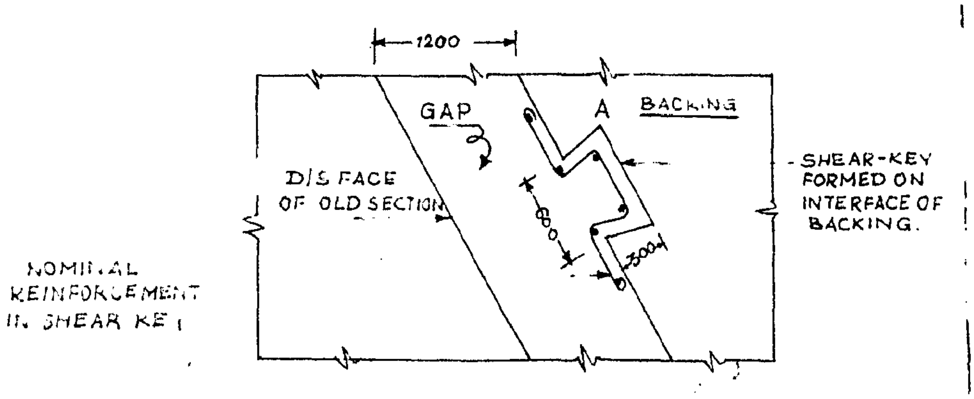
5.11.4 Shear Keys

5.11.4.1 Shear keys were provided on the downstream face of the existing section by cutting out concrete using pneumatic pavement breakers. Similar keys were formed on the interface of the new backing also while concreting.

5.11.4.2 Main reliance was kept on the bond between the two structures for resisting the shear stresses at the interface. Wherever this was not possible, steel rods were designed to take up the shear. It was thought that if all the keys did not get simultaneously mobilised, the whole force will be transferred to those in contact in the first instance and after their failure to the rest making them also fail. On these considerations, shear keys were provided only as an additional precautionary measure.

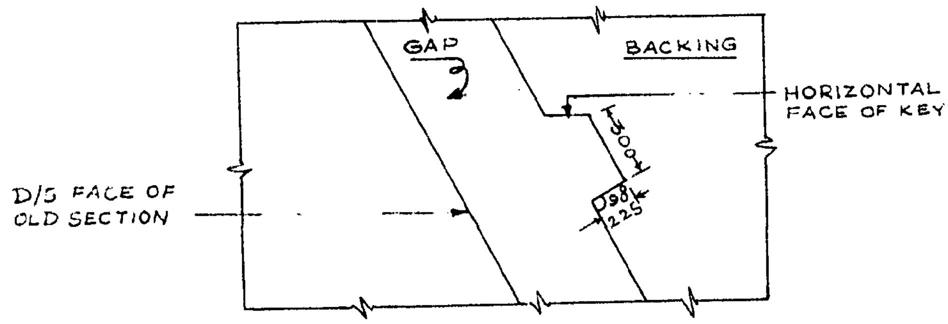
5.11.4.3 To prevent improper and out of shape cutting of keys in the downstream face of the dam, series of holes were first drilled round the periphery of the shear keys at a spacing of about 22.5 cms. This prevented overcuts as well as ensured cutting in proper shape.

5.11.4.4 In the beginning, the desired shape of the shear key was as shown below. This was favoured from the consideration that, it will physically prevent separation of the new and old concretes if the two tend to vibrate seperately during an earthquake. However this shape presented difficulty during concreting. It was very difficult to force concrete up into the corner at 'A' by vibration. The concrete was then placed at 12 mm slump.



5.11.4.5 It was thought that, by making the concrete more fluid (i.e. by increasing slump) it may be possible to fill the corner. To study the behaviour of concretes of low and high slump, a model was constructed on the field (prototype model) with perspex windows for the purpose of inspection. From these experiments, it was seen that, it was not advantageous to increase the slump (from point of view of filling the corner) beyond 50 mms. Concrete with 50 mms slump possessed body and hence it could be made to rise up in the corner by vibration. Vibration of concrete with more slump instead of pushing the concrete up tended to spread it laterally due to its extra fluidity. It was also necessary that the concrete placement was as fast as possible as concrete responded to vibration better while it was fresh. From these experiments, it was decided to retain the slump at 50 mms.

5.11.4.6 In spite of this change in slump, in actual working in was found that, it was not always possible to satisfy the above requirements. It was therefore decided first to reduce the depth of the key from 300 mm to 225 mm and then later to make the top face of the key horizontal as shown below. This modification was necessary only in the case of shear keys on the interface of the backing concrete.



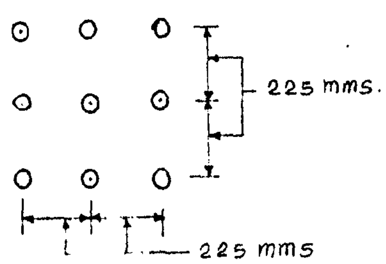
5.11.5 Reinforcement Bars :

5.11.5.1 As stated earlier, dowels were provided for bonding or holding together the two concretes as well as for resisting tensile and shear stresses also in some regions.

Where reinforcement was provided to resist tensile and shear stresses, cold twisted deformed bars were used. Use of mild steel bars would have meant too much congestion of steel in the gap which would have made placement of concrete difficult. These steel bars were arranged to coincide with the top of every lift. By such provision, they did not interfer with shuttering arrangements provided on the interface of backing concrete.

5.11.5.2 Inspite of using torsteel ribbed bars in some zones as much as 54 bars had to be provided at one level within 9 metre width only. Had the bars been equally spaced, the operations of placing gap concrete would have been greately hampered. Therefore there was no other alternative but to provide these bars in groups, each group containing from 2 to 9 bars.

5.11.5. 3 There was a possibility that if the holes for the bars in a group were drilled too close to one another, during the drilling operation itself or under the action of the pull out force, the concrete between them may crack easily and the group may come out as a whole. Also while drilling the holes, if drilled slightly out of line, they may intersect each other. Considering these possibilities, a distance of 22.5 cms was kept between the adjacent bars as shown below :-



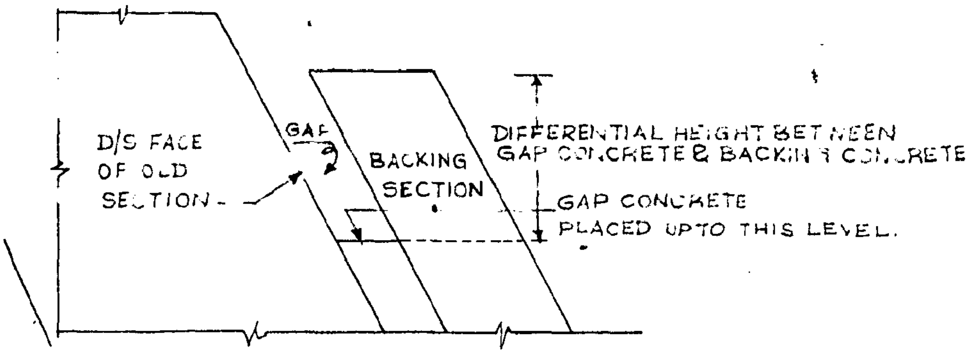
5.11.5.4 It was confirmed by actual field tests that this spacing was adequate to prevent a crack forming across the holes in the intervening thin concrete when an adequately large pull out force was applied.

Pull out tests were also conducted on individual bars to determine the embeddement lengths required as mentioned earlier.

5.11.6 Differential height between gap concrete and backing concrete:

5.11.6.1 Although the new backing concrete was built independentely of the existing dam to avoid undesirable shrinkage stresses, it neither derived its support through ribs as in the case of Mundaring weir nor through steel dowels as in the case of Asswan dam. The method followed in construction in the present case was thus different from the usual methods adopted until now.

5.11.6.2 The gap concrete in any lift was placed only when the concrete in the backing portion (laid separately) became about 45 days old, in which period its shrinkage was expected to be mostly complete. Thus there was a difference between the top levels of the gap concrete and the new backing concrete equal to the height through which the backing concrete could be raised during this 45 days period. Supposing the concrete to be laid in 1.5 metres thick lifts at the rate of one lift every 3 days, then about 18 metres of raising could be done within 45 days. However it was seen that, it was not possible to allow such a big difference in elevations of the two concretes owing to the development of tensile stresses in the backing concrete on its downstream face when the differential height exceeded a certain limit. This is evident from the Sketch below :-




5.11.6.3 In calculations of the stresses in the cantilevering backing concrete in its unsupported state, some effect of possible earthquake forces had to be provided for. Thus allowable differential heights, when gap concrete reached various levels, were worked out permitting maximum of about 2.04 kg/cm$^2$(30 p.s.i.) of tension under the effect of earthquake with horizontal acceleration of 0.1 g assumed from downstream towards upstream. This height varied from 7.5 m to 16.5 m depending upon the monolith and the level of gap concreting reached. This condition coupled with the condition of permissible reservoir water levels for placing joining concrete at various levels had to be carefully considered in planning and executing the programme of construction.

5.11.7 <u>Temperature control of concrete</u>:

5.11.7.1 The following steps were taken in connection with the temperature control.:-

i) Although ordinary portland cement was used, only that obtained from Shahabad factory was used as it was found to generate lesser heat as compared to cements from other factories received on the project.

ii) Successive lifts of concrete were placed at an interval of 5 days.

iii) Height of lift was limited to 1.5 metres for the backing section, 3 m high lifts were considered permissible for gap concrete.

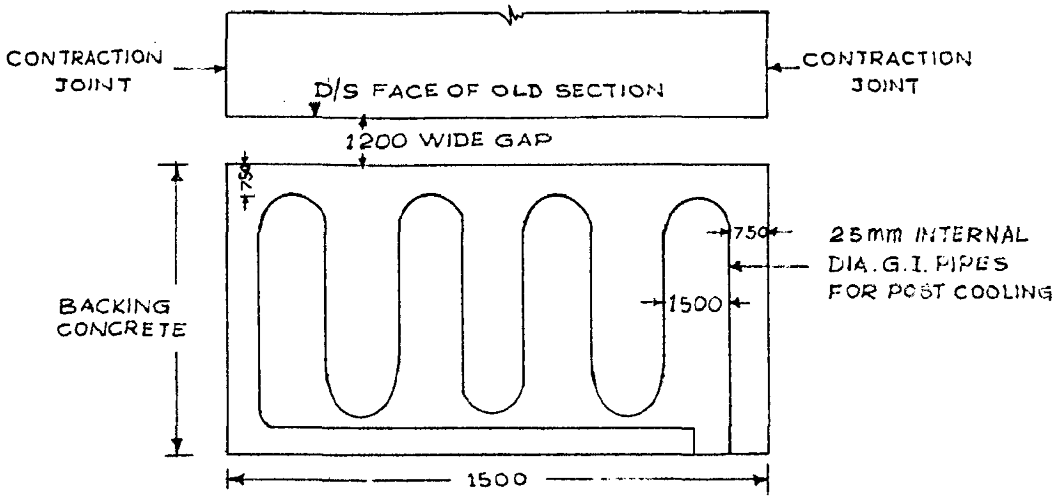
iv) Concrete was always precooled when used for filling the gap. Its temperature was generally kept at 60°F or less if possible. When used in backing, it was precooled only when atmospheric temperatures were high, i.e. in summer months of April & May.

v) Concrete was post cooled by circulation of cold water through embedded pipe system.

These measures helped in controlling the temperature rise as well as cooling the concrete to stable temperatures generally within one to one and a half months' period.

5.11.7.2 For the purpose of precooling of concrete, part of the mixing water was replaced by flaked sub-cooled ice. For this purpose, an ice plant was installed having a capacity of producing 50 tons of ice per day. This plant had 5 ice making units, each producing 10 Tons of flaked ice per day. The ice was added in the mixers in the required quantity.

5.11.7.3 For post cooling of concrete, 25 mm internal diameter G.I. pipes were provided on the top of every 1.5 metre thick lift. The horizontal spacing of these pipes was 1.5 metres. A typical layout of this arrangement is shown in the figure given below :




PLAN

Reservoir water at 22.2°C (72°F) was circulated through these pipes at the flow rate of 4 g.p.m.

5.11.7.4 The backing concrete was placed at a temperature of 23.9°C (75°F) while the gap concrete was placed at 15.6°C to 18.4° C. It was observed from the embedded resistance thermometers that, the temperature rise of backing concrete varied from 13.9°C to 19.4°C. The average annual temperature is about 29.4°C.

These arrangements permitted cooling of the backing to the required extent within about 45 days.

5.12 The above are some of the important aspects of the strengthening work executed in the case of Koyna dam. This work was mostly completed by June 1972.

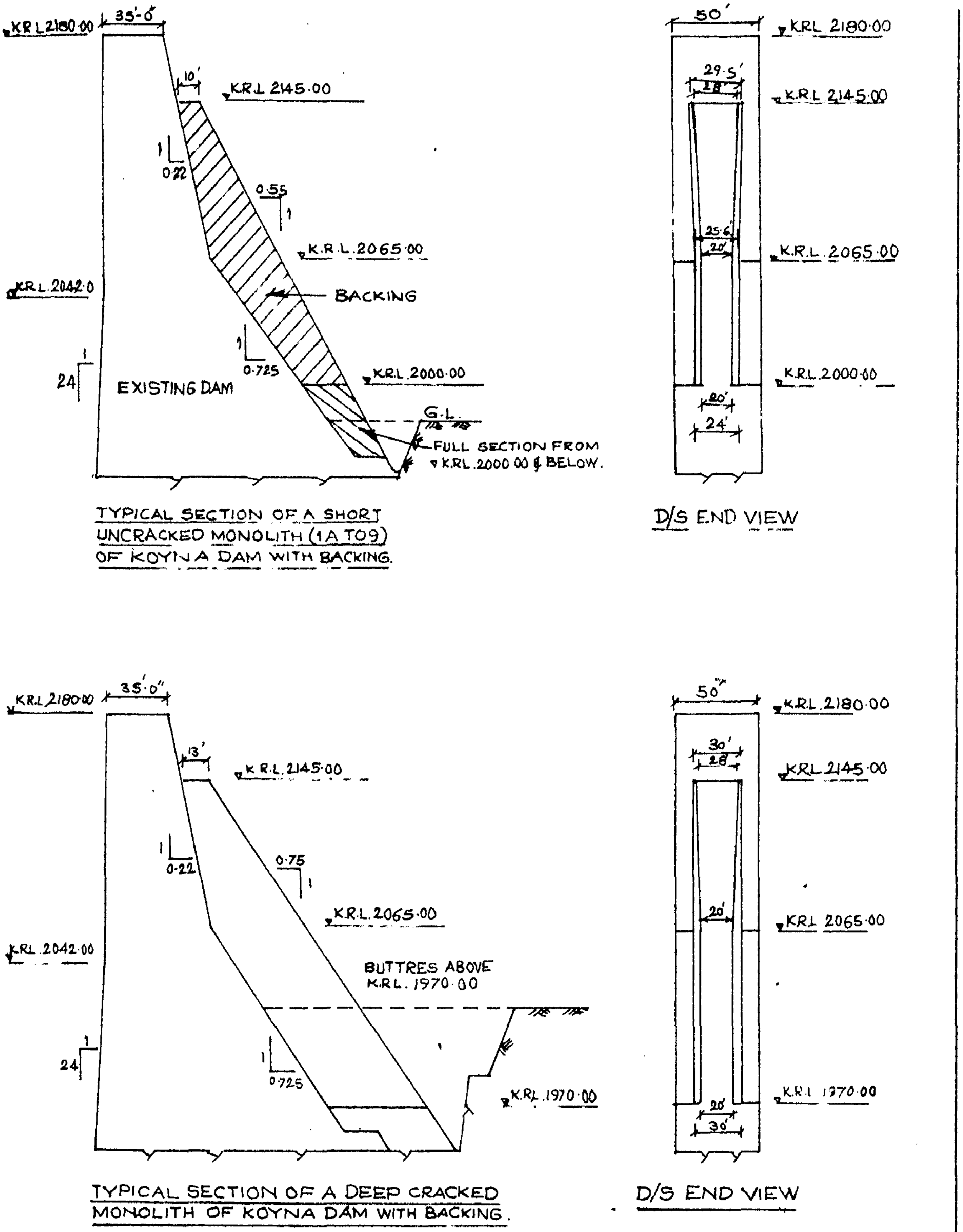
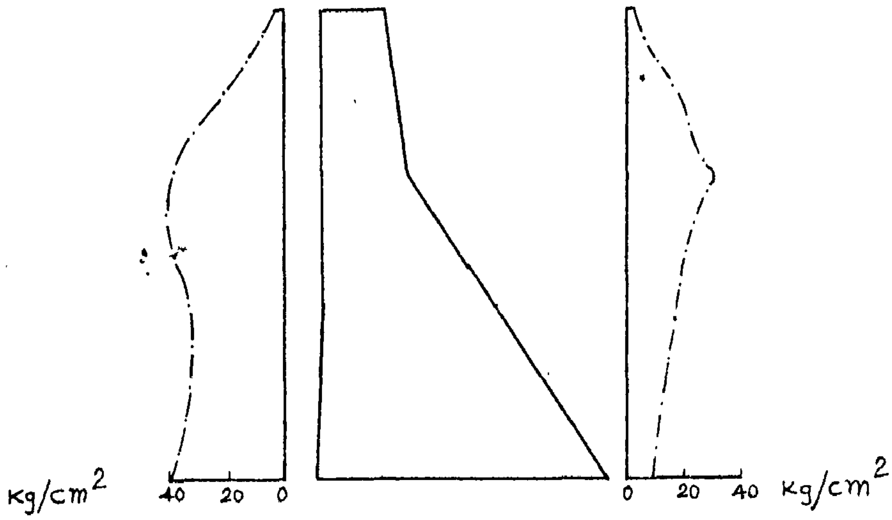


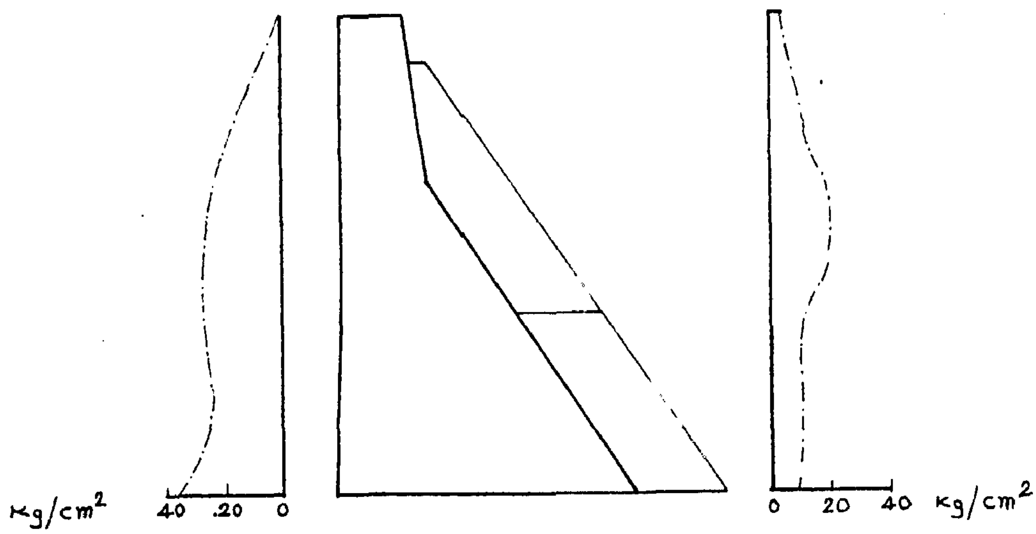

FIG 51

TYPICAL SECTIONS OF KOYNA DAM, WITH BACKING SECTIONS.




—·— DUE TO HORIZONTAL GROUND MOTION WITH RESERVOIR.

VARIATION OF VERTICAL NORMAL DYNAMIC STRESS ALONG THE UPSTREAM AND DOWNSTREAM FACES OF EXISTING NONOVERFLOW SECTION OF KOYNA DAM.




—·— DUE TO HORIZONTAL GROND MOTION WITH RESERVOIR.

VARIATION OF VERTICAL NORMAL DYNAMIC STRESS ALONG THE UPSTREAM AND DOWNSTREAM FACES OF STRENGTHENED NONO-VERFLOW SECTION OF KOYNA DAM

FIG. 5 2

DYNAMIC STRESSES IN KOYNA DAM

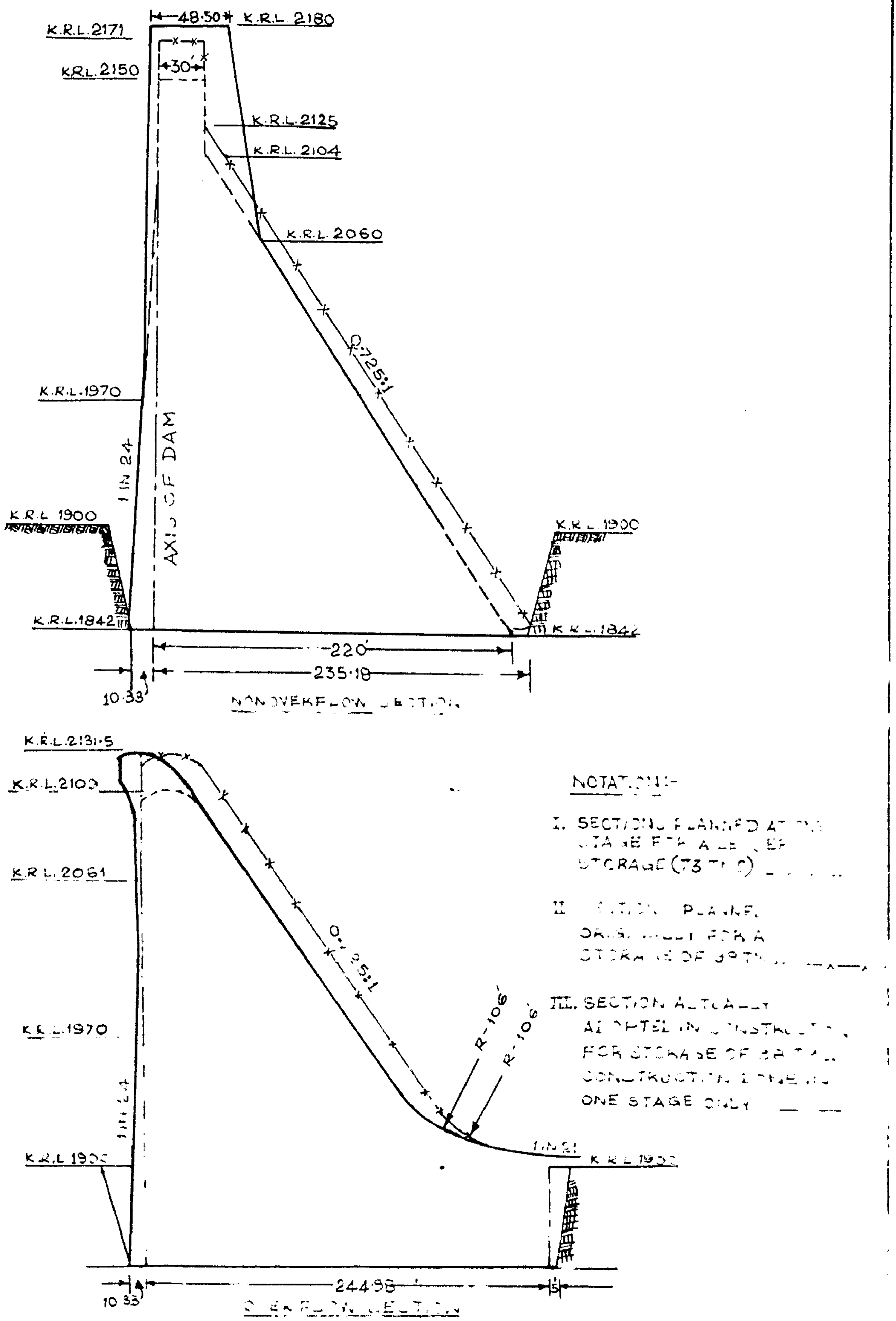
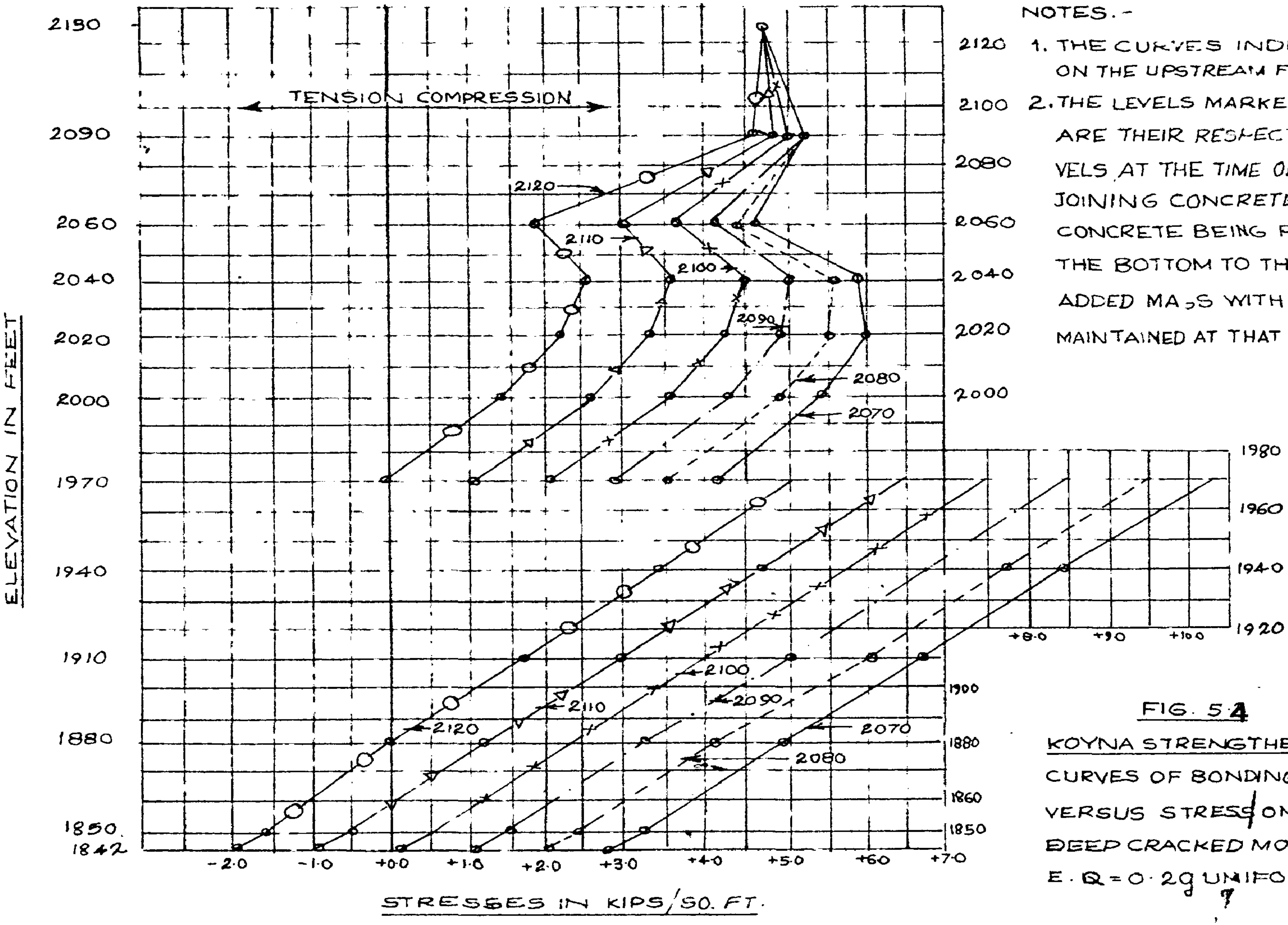



FIGURE - 5.3

KOYNA DAM SECTIONS




NOTES.-

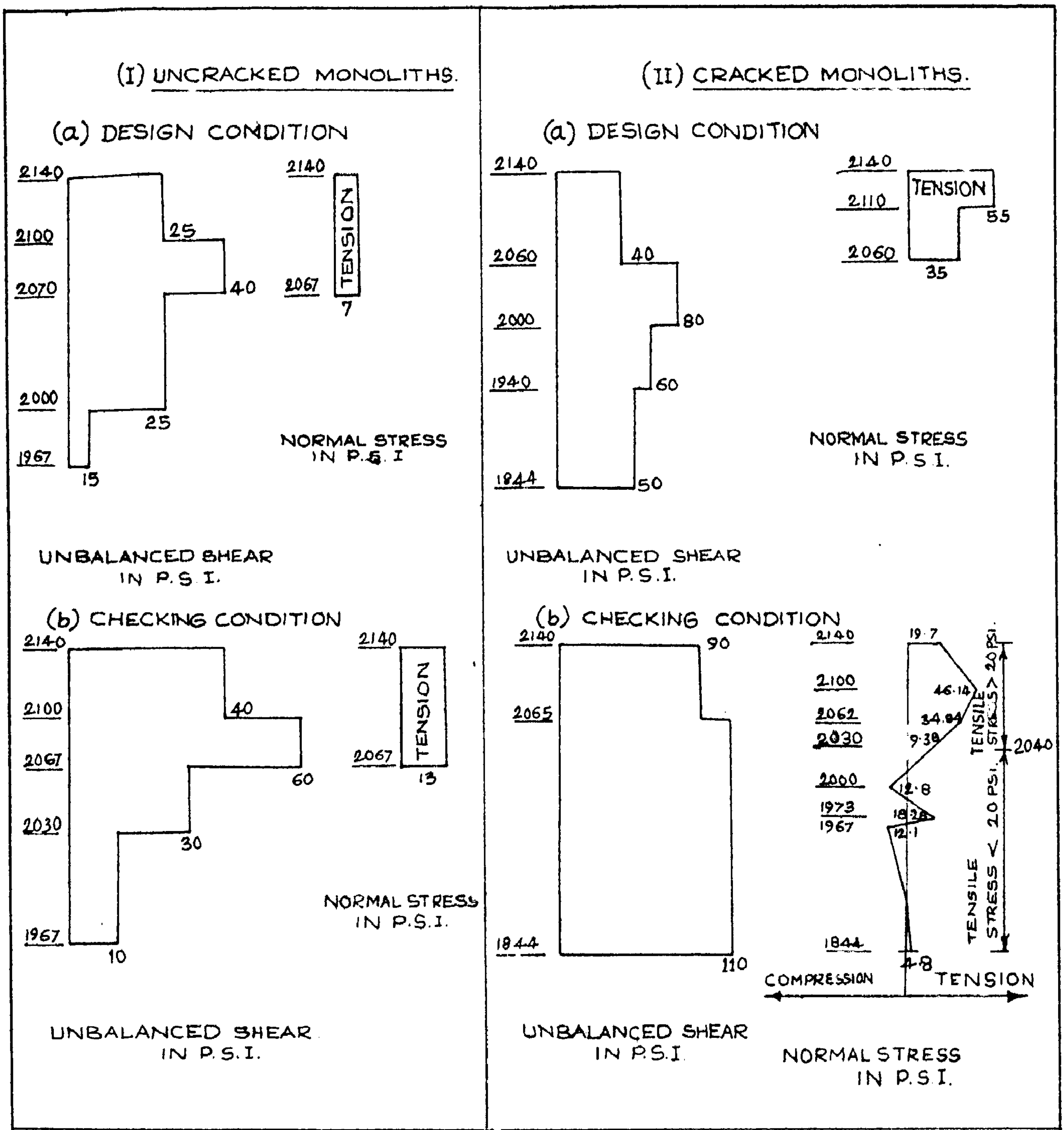
1. THE CURVES INDICATE STRESSES ON THE UPSTREAM FACE OF THE DAM.
2. THE LEVELS MARKED ON THE CURVES ARE THEIR RESPECTIVE WATER LEVELS AT THE TIME OF LAYING THE JOINING CONCRETE, THE JOINING CONCRETE BEING PLACED FROM THE BOTTOM TO THE TOP OF THE ADDED MASS WITH THE RESERVOIR MAINTAINED AT THAT LEVEL.

FIG. 5.4

KOYNA STRENGTHENING STUDIES

CURVES OF BONDING LAKE LEVEL VERSUS STRESS ON U/S FACE FOR DEEP CRACKED MONOLITHS.

E.Q = 0.2g UNIFORM.




NOTES :–

UNBALANCED SHEAR SHEAR STRESS - NORMAL STRESS X TAN $\phi$ THE VALUE OF '$\phi$' ASSUMED HERE IS 55°. THE TERM NORMAL STRESS X TAN $\phi$ IS NEGLECTED WHEN TENSION IS DEVELOPED AT THE JOINT.

FIG. 5.5

DISTRIBUTION OF SHEAR AND NORMAL STRESSES OF D/S FACE OF EXISTING SECTION OF KOYNA DAM AFTER STRENGTHENING.

# CHAPTER VI

# S U M M A R Y

# CHAPTER VI

## Summary

6.1 The most common and positive method of strengthening gravity dams, whether in connection with repairs or heightening, is the provision of downstream backing made monolithic with the existing structure to obtain monolithic structural action of the entire structure.

6.2 Monolithic condition can be ensured by proper design of the joint between the new and the old section as this joint is the weak link. Thus design of a strong and dependable joint is the crux of this problem. This very aspect is the main concern of the designer and creates lots of difficulties at the time of execution to the construction engineer. The problem becomes more complicated when shearing and tensile stresses of considerable magnitude are expected to occur at the joint.

6.3 In case of massive additions, the development of intense shearing stresses at the joint-plane and tensile stresses on the upstream face preclude the possibility of direct placement of the backing concrete against the existing section. Suitable methods of construction have therefore to be adopted which eliminate the development of such stresses, but help to achieve the design objectives. The method most widely used makes use of the provision of an initial gap of workable width between the new and the old sections which is filled subsequently by jointing concrete.

6.4 The effect of reservoir operation on the construction schedule has to be carefully considered while bonding the new section with the old. Indiscriminate bonding can result in tensile stresses on the upstream face under the final conditions of loading.

6.5 Temperature control of concrete also needs particular attention. Various measures right from the selection of the type of cement to the post cooling arrangements require careful consideration. As the bonding of the two sections can be effected only when the new concrete has attained stable temperature conditions, the period of cooling of the new backing section needs to be suitably controlled so as to enable non-stop placement of concrete.

6.6 If the heightening is forseen, some provisions can be made in the design of the temporary section, so that the cost of special measures required to ensure proper bonding between the two sections is kept to the minimum.

6.7 Thus strengthening of gravity dams involves problems both in design and construction, which demand ingenuity on the part of the designer and utmost care on the part of the constructor to realise the assumptions made in the design. With the need for exhaustive utilisation of water resources and best use of the limited funds, heightening and therefore strengthening of dams has no doubt assumed greater significance in the days to come.

# CHAPTER VII

# C O N C L U S I O N S

## CHAPTER VII

### Conclusions :

7.1 Looking to the need for utilisation of every drop of water gifted by nature in an economical manner, it has become necessary to heighten some of the existing dams built in the past. These dams had been then built to eater for the demands of the immediate future or according to the financial resources available then and hence there exists a scope for increasing their storage capacity by heightening economically instead of attempting the exploitation of costly unfavourable sites.

7.2 Secondly there are old dams designed and constructed on the basis of old design practices, which now require buttressing if they are to be made safe in accordance with the present day design standards. This has necessitated strengthening of some of the old dams today. So also due to revisions in the seismic mapping of the country due to some recent earthquakes, dams already built might need strengthening in some regions of India.

7.3 Among the various methods of strengthening gravity dams, addition of monolithic concrete backing on the downstream side is considered to be the one that is the most reliable and a positive method of strengthening.

7.4 For comparatively low heightenings and in case of dams of small height, the method of prestressed cabling may be more suitable and economical as compared to the provision of downstream-backing. In case of appreciable raisings and dams of moderate to large heights, however, downstream backing appears to be the only reliably acceptable solution. Prestressed cabling can be however very well used in conjunction with the stage construction where its durability is not imperative and its advantage can be taken in the reduction of the temporary section as well as for adjusting the stress distribution in the structure to the desired status at the time of resumption of construction. However in seismic zones, where the structure is likely to be subjected to the action of earthquake forces & the regulation of the reservoir is such as to cause variation of the lake level through a large range, use of prestressed cables will not be a satisfactory method to adopt.

7.5 Upstream backing involves emptying of the reservoir for a considerable period of time and thereby means loss of revenue and production which is so vitally needed for the country's economy. Therefore it can not be considered in lieu of downstream backing except possibly when the material of the existing structure is very poor for effecting a strong bond between the old and the new work. However unbonded downstream backing could still be considered in preference to upstream backing.

7-6 Provision of earth backing on the downstream can be considered only in aseismic zones and where economics and availability of required construction materials favour its use. However when the existing structure is of masonry or concrete, it is a pointer to the availability of rock as the most economically available construction material in the vicinity of the site. So also utilization of only downstream quarries will be involved in case of provision of earth backing, which may involve costly land aquisition. On the contrary, the rock quarries earlier used for the construction of the existing structure will not present any such problems if suitably located. For low reservoir levels, tensile stresses may develop on the downstream face of the dam section on account of the downstream earth backing. Therefore downstream earth-backing will be suitable in case of those dams having such reservoir regulation that the variation in the lake level is small. In seismic zones, the effectiveness of the downstream earth backing is rather doubtful during an earthquake.

7.7 Although provision of monolithic downstream backing involves problems, these can be overcome by proper design and careful execution. The numerous examples of strengthening carried out in the world uptil now can be cited for justifying such a confidence in this method of strengthening gravity dams.

7.8 Considering the need for heightening and strengthening of existing structures constructed in the past, the need of water for rapid development on all fronts, justification for sharing of the financial burden of development by humans in all ages and the economies involved, stage construction needs to be planned in case of projects in hand which are in the design or the planning stage. This will incidentally minimise the cost which has to be incurred in case of unforseen strengthening.

7.9 It may therefore be concluded that, the method of successively thickening the gravity section for the economic development of water resources may be regarded as generally the most suitable and dependable method for the purpose of rendering positive support or for the creation of a single monolithic structure which can behave as if it had been originally built to its final dimensions. It may even be said that, on account of the greater ease with which subsequent raisings can be done in the case of gravity dams when required, in some cases concrete or masonry dams may be adopted in preference to earth dams from this point of view.

7.10 <u>Need of compilation of data for analysis</u>

7.10.1 Although several examples can be cited of provision of downstream backing for strengthening gravity dams, very little information is available as regards the results and conclusions about the actual behaviour of the strengthening section on the basis of instruments embedded inside the mass. Thus it is very important to know the temperature generated in the original dam, the backing slab and slot filling, the shrinkage movement of the backing slab in relation to the original dam while it is free and the strain in the gap concrete which might be caused either by its own shrinkage or by further shrinkage in the backing slab after filling the gap. Measurements can also be made by the use of strain gauges at the top of the gap filling of the possible movement between the gap concrete and the original dam and between the gap filling and the new backing under the effects of external loads. It is necessary that, such data in case of dams similarly strengthened is compiled and useful conclusions arrived at, which can provide information of use in case of further strengthening works.

7.10.2 While estimating the stresses in the existing section, especially the locked up stresses, some assumptions as regards the effect of the separately constructed backing on stress distribution in the existing section have to be made, as direct evaluation of such effect is not possible. Validity of such assumptions needs to be checked in each particular case on the basis of systematic and planned observations of instruments in the existing section and the analysis of such observations. On the basis of such studies, realistic design criteria can be evolved.

## : LIST OF REFERENCES :


1. J. Toran, General Report - VIth I. C. O. L.D., Volume-I, New York-1958.
2. Dr.K.L.Rao, Strengthening of Masonry Dams, Irrigation & Power, Journal of C.B.I.P., India, Vol.10, January 1953.
3. Dr.Ing.Cristea Mateeseu, Etude Synoptique Des Dispositions De Surelevation Des Barrages En Beton, VIth I.C.O.L.D., Volume I, N.Y. 1958.
4. Dr.K.L.Rao, S.K.Dhawan, Stage Construction of Dams in India-Its economies and difficulties, VIth I.C.O.L.D., Volume I, New York, 1958.
5. F.Groner, Heightening of existing dams including methods of new dams in successive stages, VIth I.C.O.L.D., Volume I, 1958.
6. B.V.Kanchi, Use of the prestressing technique in construction of concrete dams with special reference to raising of existing dams, M.E.Dissertation accepted by University of Roorkee, October 1970.
7. R.Priscu & M.Constantinescu, Considerations Concernant La Surelevation Et La Construction En Etapes Des Barrages - Poids, Par Precontrainte, VIth I.C.O.L.D., Volume I, New York, 1958.
8. E.R.Dexter and F.A.Houck, Design and Construction Considera tions in Heightening Concrete Dams, VIth I.C.O.L.D. Volume I, New York, 1958.
9. A Bourgin, Book - "The design of dams".
10. E.Becerril, Heightening of existing dams including methods of constructing new dams in successive stage, VIth I.C.O.L.D. Volume I, New York, 1958.

11. Dr.H.Zaki, Heightening of the Asswan Dam, VIth I.C.O.L.D., Volume I, New York, 1958.

12. S.Leliavsky Bey, Stresses and temperature effects in dams with particular reference to Asswan dam, Journal of the Institution of Civil Engineers, No.3, January 1948.

13. M.Mc Donald, Asswan Dam : Protection of downstream rock surface and thickening and heightening, Minutes of Proceedings, Instr. Civil Engineers, Vol.CxCIV, 1912-1913, Part IV.

14. "Raising O'shaughnessy Dam", Indian Concrete Journal, October 1939.

15. S.W.Stewart, Experience of the Ambursen Engineering Corporation in raising the height of dams, VIth I.C.O.L.D., Vol.I, New York, 1958.

16. "Heightening of Lages Dam", Riode Janeiro, Engineering, Vol.168, July 1949.

17. Bogle J.M.L., Ross R.M., Me Millan T, Design & Construction of Avon dam, Journal of Institution of Civil Engineers (London), Vol.12, January 1959.

18. Discussion and Author's reply on "the design & construction of Avon dam", Journal of Institution of Civil Engineers (London), Vol.13, August 1959.

19. A.Presmanes, Surelevation Du Barrage De Doiras, VIth I.C.O.L.D., Vol.I, New York, 1958.

20. M.Kondo & M.Kakitani, Plan of heightening Odomari dam, VIth I.C.O.L.D Volume I, New York, 1958.

21. C.M.Roberts, The heightening of a gravity dam, Vth I.C.O.L.D., Volume-II, Paris, 1955.

22. Kanwair Sen, "Modern trends in design and construction of dams & Power Houses".

23. V.C.Munt, Design for the raising of Mundaring Weir, Inst. Engrs. Australia Journal, Volume 20, Nov.1948.

24. H.Press, Heightening of Dams, VIth I.C.O.L.D.,Volume I, New York, 1958.

25. A.Z.Bassevitch, Stress distribution in building and in heightening of concrete dams, VIth I.C.O.L.D.,Volume I,New York,'58.

26. F.G.Thomas, Structural Engineering, Vol.14(7),1936.

27 . L.H.Dickerson, Design and construction of hydro-electric dams built in Scotland since 1945, Journal of Institution of Civil Engineers (London),Vol.29, Dec.1964.

28. Discussion and Author's reply on 'Design and construction of hydro-electric dams built in Scotland since 1945', Journal of Inst.of Civil Engineers (London), Volume-33, March 1966.

29. Robert G. Minchin, Special Features of the Affric Hydro-electric Scheme, Journal of Institution of Civil Engineers (London), Paper No.5926, 1953.

30. M.D.Copen & E.A.Lindholm, Replacing East Canyon Dam while operations continue, Journal of Power Division Proc. of A.S.C.E., March 1967.

31. Leliavsky S., Book -"Irrigation & Hydraulic Design - Vol.III" Chapman & Hall Ltd. (London).

32. G.M.Munday, Burrinjuck, Dam - Strengthening and Enlargement, Institution of Engineers Australia Journal,Vol.27, June 1956-

33. "Marshall Ford dam designed for raising in future", Engineering News Record, May 1937.

34. "Enlargement of Alpine Dam", World Construction News, Dec.1941.

35. B.G.Dholkia and P.K.Nagarkar, Stage Construction of High Dams, Indian Concrete Journal, April 1958.

36. "Mulholland dam backed by earthfill", Engineering News-record, May 1934.

37. Rawhouser, Cracking and temperature control in mass concrete, A.C.I.Journal, February 1945.

38. B.Hellstrom, Decay and Repair of concrete and masonry dam, Structural Engineer, May 1933.

39. W.P.Creager, A.D.Justin & J.Hinds, "Book-Engineering for Dams - Volume II", John Wiley and Sons.

40. S.S.Saini, Jai Krishna & A.R.Chandrasekaran, Earthquake Stress analysis of Koyna dam monoliths, an unpublished report.

41. "Entrained air - A factor in the design of concrete mixes", Materials Laboratories, Report No.C-310, U.S.B.R., March 1946.

42. A.M.Neville, Book -"Properties of Concrete".

43. J.Wadell, Book- "Concrete Construction Handbook".

44. Report of the Committee of Experts appointed by Government of India in connection with Koyna earthquake, Volume I, 1968.

45. V.R.Deuskar & E.D.Shende, Koyna in retrospeet, The Maharashtra Public Works Journal, Vol.III, Jan.-June-1962.

46. Jai Krishna, A.S.Arya & S.K.Thakkar, Strengthening of Bhatgar Dam against seismic forces by earth backing on downstream face, an unpublished report of Sept.'71.

47. U.Langefors & B.~~Kxxxx~~ Kihlstrom, Book - "The modern technique of rock blasting", John Wiley & Sons.

48. C.L.Emmerling, Montana Power Company's Dam repair Program, Journal of construction division, Proc. of A.S.C.E., September 1972.

49. N.G.K.Murti, P.M.Mane, V.R.Deuskar, Koyna earthquake -Remedial measures to cracks in dam, Xth I.C.O.L.D., Montreal, June 1970.

50. Saini S.S., Jai Krishna and A.R.Chandrasekharan, Behaviour of Koyna Dam in Dec.11,1967 Earthquake, Journal of Structural Division, A.S.C.E., Vol-98, No.ST 7, July 1972.

51. Specifications for the work of "Strengthening of Koyna Dam by Concrete backing" .

52. Unpublished Literature regarding the work of "Strengthening of Koyna Dam."