

Design Criteria for Concrete Arch and Gravity Dams

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Design Criteria for Concrete Arch and Gravity Dams

Office of Design and Construction
Engineering and Research Center
Denver, Colorado 80225



United States Department of the Interior
Bureau of Reclamation



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Preface

This revision of Engineering Monograph No. 19 presents current Bureau of Reclamation design criteria upon which are based design decisions concerning mass concrete dams. The basic considerations and specific design criteria set forth in this monograph constitute the present-day standards for Bureau designs.

More than two decades have elapsed since the first issuance of this monograph in 1953. The intent of the publication at that time was to document basic concepts bearing on Bureau of Reclamation designs of concrete arch and gravity dams that had evolved during the preceding 50 years. Compilation of the criteria that were essential to successful construction and operation of many notable concrete dams on Reclamation water resource projects, such as Hoover, Grand Coulee, Shasta, and Hungry Horse Dams, resulted in the first orderly record of the Bureau's philosophy and basic criteria affecting the conception and development of concrete dams. The Bureau's recognition of the importance of keeping abreast of changing design technology was evidenced by revision of the monograph in 1960.

In the nearly 15 years following the first revision of the monograph, design technology advanced at a rapid rate. Such new knowledge as finite element

analysis, geologic advances leading to increased understanding of foundations and abutments of dams, improved techniques of seismic analyses, development of computer techniques for trial-load analyses, and other advances accompanied by the progress in computer technology opened new avenues to design and, in turn, brought about refinements in the basic concepts guiding dams design. Dams completed during this period, including Glen Canyon, Flaming Gorge, Morrow Point, and Yellowtail Dams, reflect this progress.

Concrete dams on Bureau of Reclamation projects continue to be key elements in project development, as indicated by such major structures as Grand Coulee Forebay, Crystal, Nambé Falls, and Auburn Dams that are under construction or in advanced stages of design.

Assurance that Bureau design practices for concrete dams remain up-to-date and consistent with currently accepted good engineering practice can be achieved only through a program of continuing review, evaluation, modification of standards, and development of new criteria as required. The review must be made in light of developing technology in all fields associated with concrete dam design. In this effort, the Bureau of Reclamation is continuing

its investigations to develop new knowledge and to help resolve as yet unanswered questions.

Following its introduction, this monograph is divided into Part I, Arch Dams, and Part II, Gravity Dams. Within Parts I and II, each criterion is discussed under its appropriate subject heading. Each discussion is composed of two parts, the basic considerations and the criterion statement. The basic considerations are intended to supply the reader with some brief background to introduce the subject and support and help explain the criterion statement.

The reiteration of certain common information in Parts I and II obviates the necessity of the reader having to refer back and forth between the two parts.

The review and evaluation of the 1960 edition of Monograph No. 19, the development of new criteria, and the technical writing of this edition of the monograph were accomplished by the engineers in the Analytical Design and Performance Unit of the Concrete Dams Section in the Hydraulic Structures Branch of the Division of Design.

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Introduction

Design Philosophy

The Bureau of Reclamation's philosophy of concrete dam design is founded on rational and consistent criteria which provide for safe, economical, functional, durable, and easily maintained structures. It is desirable, therefore, to establish, maintain, and update design criteria. Under special conditions, consideration can be given to deviating from these standards. In those situations the designer bears the responsibility for any deviation and, therefore, should be careful to consider all ramifications. Accordingly, each of the criteria definitions in this monograph is preceded by a discussion of the underlying considerations to explain the basis of the criterion. This serves as a guide in appraising the wisdom of deviating from a particular criterion for special conditions.

Loadings.—The designer can assure the safety of concrete dams by designing for all combinations of loads including those whose simultaneous occurrence is highly improbable and by using unduly large safety factors. This may lead to overly conservative, uneconomical designs. A structure designed for the loading combinations and corresponding safety factors listed in this monograph should be safe, yet economical.

Design Data.—Modern methods of analysis are powerful and sophisticated; yet, without meaningful and accurate input data, they may produce useless and even erroneous results. Design data must be determined as accurately and completely as possible. The data should be derived from field and laboratory tests plus measurements taken from in-service dams. For occasional situations where the data are incomplete, the designer may supplement these data by referring to other dams with similar conditions. When data are absolutely unavailable, he may estimate them by using conservative engineering judgment. This approach may lead to overly conservative designs and emphasizes the advantage of conducting comprehensive programs to obtain adequate design data.

Safety Factors.—Safety factors have been established to limit the allowable stresses in the materials as determined by analysis. The need for safety factors is due primarily to uncertainties in (1) the service loads, (2) the variability of materials, (3) construction practices, and (4) the correctness of analyses. These uncertainties also preclude determining the safety of the dam exactly; therefore, factors of safety are selected based upon experience and judgment.

In addition to the safety factors specified, additional allowances for safety tend to enter into a design as a consequence of overly conservative treatment of successive uncertainties encountered in sequential design operations. Structures so designed may possess margins of safety in excess of the designer's intent and are uneconomical. The safety factors recommended herein are considered to provide for all underlying uncertainties and should be used without additional provision for safety, except under conditions of unusual uncertainty or hazard. For such conditions, some additional margin of safety may be provided by a judicious increase in the appropriate safety factor.

The general philosophy of safety of dams is that the magnitude of safety factors can be reduced as the probability of occurrence for particular loading conditions decreases. Also, as methods of analysis are improved and conditions associated with the safety factors are more accurately determined, the safety factors should be reduced. Although some structural damage may be foreseen under certain extreme loading conditions, damage resulting from sudden release of the reservoir must be prevented.

Analytical Basis.—Methods of analysis used to design arch- and gravity-type concrete dams should

be the best available. The considerations used to develop the criteria in this monograph were based on the Bureau of Reclamation's analytical methods, which are described in references [1] and [2].¹ Therefore, it is recommended that these criteria not be used in conjunction with other analytical methods without careful consideration.

Construction Quality.—The methods of analysis and design criteria used by the Bureau of Reclamation have been developed on the assumption that dams will be constructed with concrete having uniformly good quality. Good concrete can be assured only through careful attention to quality control. Many factors enter into quality control: (1) testing and inspection of materials selected for use, (2) proper proportioning and adequate mixing of the materials, (3) use of proper handling, placing, and consolidating procedures, (4) proper preparation of placing surfaces, and (5) proper curing.

The best of materials and design practices will not be sufficiently effective unless the actual construction practices and procedures are properly performed. Building modern, well-designed concrete dams imposes upon those in charge of construction work the responsibility for assuring that the concrete is of uniformly good quality.

¹ Numbers in brackets refer to items in the bibliography.

Part I Arch Dams

Bureau of Reclamation

Arch Dams

CONCRETE PROPERTIES—STATIC

Strength

Basic Considerations.—An arch dam should be constructed of concrete that meets the design criteria for strength, durability, permeability, and other required properties. Because of the sustained loading generally associated with them, the concrete properties used for the analyses of static loading conditions should include the effects of creep. Properties of concrete vary with age, the type of cement, aggregates, and other ingredients, as well as their proportions in the mix [3]. Since different concretes gain strength at different rates, measurements must be made of specimens of sufficient age to permit evaluation of ultimate strengths.

Although the concrete mix is usually designed only for compressive strength, appropriate tests should be made to determine the tensile and shear strength values.

The mix should be proportioned to produce concrete of sufficient strength to meet the design requirements. Concrete strengths should be determined by tests of the full mass mix in cylinders of sufficient size to accommodate the largest size aggregate to be used. The compressive strength of concrete, determined as specified above, should satisfy early load and construction requirements and

at some specific age should have a ratio to the allowable working stress as determined by the designer.

The specific age is often 365 days, but it may vary from one structure to another. The strength should be based on an evaluation of ultimate strength and safety factor requirements discussed later in these criteria.

Tensile strength of the concrete mix should be determined as a companion test series to the tests for the compressive strength.

Shear strength is a combination of cohesive strength and internal friction which varies with the normal compressive stress. Companion series shear strength tests should be conducted at several different normal stress values covering the range of normal stresses to be expected in the dam. These should then be used to obtain a curve of shear strength versus normal stress for test cylinders of the same age as required for compressive and tensile test cylinders.

Elastic Properties

Basic Considerations.—Poisson's ratio, the sustained modulus of elasticity of the concrete, and the latter's ratio to the deformation modulus of the foundation have significant effects on stress distribu-

tion in the structure. Values of the modulus of elasticity, although not directly proportional to concrete strength, follow the same trend with the higher strength concretes having a higher value for modulus of elasticity. As with the strength properties, the elastic modulus is influenced by mix proportions, cement, aggregate, admixtures, and age. The deformation that occurs immediately with application of load depends on the instantaneous elastic modulus. The increase in deformation which occurs over a period of time with a constant load is the result of creep or plastic flow in the concrete. The effects of creep are generally accounted for by determining a sustained modulus of elasticity of the concrete for use in the analyses of static loadings.

Instantaneous moduli of elasticity and Poisson's ratios should be determined for the different ages of concrete when the cylinders are initially loaded. The sustained modulus of elasticity should be determined from these cylinders after specific periods of time under constant sustained load. These periods of loading are often 365 and 730 days. The cylinders to be tested should be of the same size and cured in the same manner as those used for the compressive strength tests. The values of instantaneous modulus of elasticity, Poisson's ratio, and sustained modulus of elasticity used in the analyses should be the average of all test cylinder values.

Thermal Properties

Basic Considerations.—The effects of temperature change in an arch dam are often a major part of the design considerations. Especially in the smaller dams, the stresses caused by temperature changes can be larger than those from the reservoir loading. The effects of temperature change depend on the thermal properties of the concrete. These properties are the coefficient of thermal expansion, thermal conductivity, specific heat, and diffusivity [4].

The coefficient of thermal expansion is the length change per unit length for 1 degree temperature change. Thermal conductivity is the rate of heat conduction through a unit thickness over a unit area of the material subjected to a unit temperature difference between faces. The specific heat is defined as the amount of heat required to raise the temperature of a unit mass of the material 1 degree. Diffusivity of concrete is an index of the facility with which concrete will undergo temperature change. The diffusivity is calculated from the values of the specific heat, thermal conductivity, and density [4].

Criteria.—Appropriate laboratory tests should be made of the design mix to determine all concrete properties.

CONCRETE PROPERTIES—DYNAMIC

Strength

The Reclamation laboratory is presently testing the strength of concrete when the concrete is subjected to dynamic loading. However, data are not yet available to indicate what the strength characteristics are under dynamic loading.

Elastic Properties

Until dynamic modulus information is available, the instantaneous modulus of elasticity determined for concrete specimens at the time of initial loading should be the value used for analyses of dynamic effects.

AVERAGE PROPERTIES

Basic Considerations.—Necessary values of concrete properties may be estimated from published data for preliminary studies until laboratory test data are available [5]. Until long-term tests are made to determine the effects of creep, the sustained modulus of elasticity should be taken as 60 to 70 percent of the laboratory value for the instantaneous modulus of elasticity.

Criteria.—If no tests or published data are available, the following average values for concrete properties may be used for preliminary designs:

Compressive strength—3,000 to 5,000 lbs/in² (20.7 to 34.5 MPa)

Tensile strength—5 to 6 percent of the compressive strength

Shear strength:

Cohesion—about 10 percent of the compressive strength

Coefficient of internal friction—1.0

Poisson's ratio—0.2

Instantaneous modulus of elasticity— 5.0×10^6 lbs/in² (34.5 GPa)

Sustained modulus of elasticity— 3.0×10^6 lbs/in² (20.7 GPa)

Coefficient of thermal expansion— $5.0 \times 10^{-6}/^{\circ}\text{F}$ ($9.0 \times 10^{-6}/^{\circ}\text{C}$)

Unit weight—150 lbs/ft³ (2402.8 kg/m³)

FOUNDATION PROPERTIES

Deformation Modulus

Basic Considerations.—Foundation deformations caused by loads from the dam affect the stress distributions within the dam. Conversely, response of the dam to external loading and foundation deformability determines the stresses within the foundation. Proper evaluation of the dam foundation interaction requires as accurate a determination of foundation deformation characteristics as possible.

Although the dam is considered to be homogeneous, elastic, and isotropic, its foundation is generally heterogeneous, inelastic, and anisotropic. These characteristics of the foundation have significant effects on the deformation moduli of the foundation. The analysis of an arch dam should include the effective deformation modulus and its variation over the entire contact area of the dam with the foundation.

The deformation modulus is defined as the ratio of applied stress to elastic strain plus inelastic strain and should be determined for each foundation material. The effective deformation modulus is a composite of deformation moduli for all materials within a particular segment of the foundation.

Foundation investigations should provide information related to or giving deformation moduli and elastic moduli. The information includes elastic modulus of drill core specimens, elastic modulus and deformation modulus from in situ jacking tests, deformation modulus of fault or shear zone materials, and logs of the jointing that occurs in recovered drill core. Information on the variation of materials and their prevalence at different locations along the foundation is provided by the logs of drill holes and by tunnels in the foundation. Good compositional descriptions of the zones tested for deformation modulus and adequate geologic logging of the drill cores permit extrapolation of results to untested zones of similar material.

Criteria.—The following foundation data should be obtained for the analysis of an arch dam:

- The deformation modulus of each type of material within the loaded area of the foundation. The effects of joints, shears, and faults obtained by direct (testing) or indirect (reduction factor) methods [10].
- An effective deformation modulus, as determined when more than one type of material is present in a foundation.
- The effective deformation modulus, as deter-

mined at enough locations along the foundation contact to provide adequate definition of the variation in deformability and to permit extrapolation to untested areas when necessary.

Shear Strength

Basic Considerations.—Resistance to shear within the foundation and between the dam and its foundation depends upon the cohesion and internal friction inherent in the foundation materials and in the bond between concrete and rock at the contact with the dam. These properties are determined from laboratory and in situ tests. The results of laboratory triaxial and direct shear tests, as well as in situ shear tests, are generally reported in the form of the Coulomb equation:

$$R = CA + N \tan \phi, \text{ or}$$

$$\begin{aligned} \text{(Shear resistance)} &= \text{(unit cohesion times area)} \\ &+ \text{(effective normal force times} \\ &\quad \text{coefficient of internal friction)} \end{aligned}$$

which defines a linear relationship between shear resistance and normal load. The value of shear resistance obtained as above should be limited to use for the range of normal loads used for the tests.

Although this assumption of linearity is usually realistic for the shear resistance of intact rock over the range of normal loads tested, a curve of shear resistance versus normal load should be used for materials other than intact rock. The shear resistance versus normal load relationship is determined from a number of tests at different normal loads. The individual tests give the relationship of shear

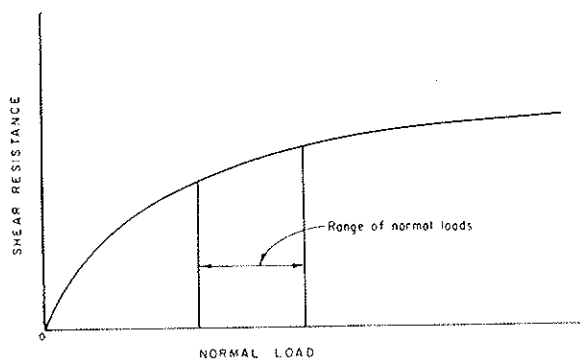


Figure 1.—Shear resistance on an existing joint in rock foundation of an arch dam.

resistance to displacement for a particular normal load. The results of these individual tests are used to obtain a shear resistance versus normal load curve as shown on figure 1. The displacement used to determine the shear resistance is the displacement that can be allowed on the possible sliding plane without causing unacceptable stress concentrations within the dam. Since specimens tested in the laboratory or in situ are small compared to the foundation, the scale effect should be carefully considered in determining the values of shear resistance to be used.

When a foundation is nonhomogeneous, the possible sliding surface may consist of different materials. Intact rock reaches its maximum break bond resistance with less deformation than is necessary for fractured materials to develop their maximum frictional resistances. Therefore, the shear resistance developed by each fractured material depends on the displacement of the intact rock part of the surface. If the intact rock shears, the shear resistance of the entire plane is equal to the combination of the sliding frictional resistance for all materials along the plane.

Criteria.—An adequate number of tests, as determined by the designer for each material along the possible sliding planes, should be made to obtain a shear resistance versus normal load relationship. The value of shear resistance recorded during tests should be measured at displacements that correspond to those expected to occur along the in situ potential sliding planes.

When the foundation along the plane of potential sliding is nonhomogeneous, the total shear resistance is the summation of shear resistances of all the materials along the plane.

Pore Pressure and Permeability

Basic Considerations.—Analysis of a dam foundation requires knowledge of the hydrostatic pressure distribution in the foundation. Permeability is controlled by the characteristics of the rock type, the jointing systems, the shears and fissures, fault zones, and, at some dam sites, by solution cavities in the rock. The exit gradient for shear zone materials that surface near the downstream toe of the dam should also be determined to check against the possibility of piping [1].

Laboratory values for permeability of sample specimens are applicable only to the portion or portions of the foundation that they represent.

Permeability of the aforementioned geologic features can best be determined by in situ testing. The permeabilities obtained are used in the determination of pore pressures for analyses of stresses, stability, and piping. Such a determination may be made by several methods including two- and three-dimensional physical models, two- and three-dimensional finite element models, and electric analogs.

If foundation grouting and drainage or other treatment are to be used, their effects on the pore pressures should be included.

Criteria.—A sufficient number of tests, as determined by the designer, should be made to determine the permeability of the foundation rock, joints and fissures, fault zones, and solution cavities [6,7].

An adequate method of analysis should be used to determine pore pressures within the foundation. The effects of any grouting, drainage, and other foundation treatment should be included.

Treatment

Basic Considerations.—Foundation treatment is used to correct deficiencies and improve physical properties by grouting, drainage, excavation of inadequate materials, reinforcement, and backfill with concrete. Some reasons for foundation treatment are: (1) improvement of deformation moduli, (2) prevention of sliding of foundation blocks, (3) prevention of relative displacement of foundation blocks, (4) prevention of piping and reduction of pore pressures, and (5) provision of an artificial foundation in the absence of adequate materials.

Regardless of the reason for the foundation treatment, its effects on the other foundation properties should be considered in the analyses.

Criteria.—Effects of treatment on foundation properties should be considered.

Compressive and Tensile Strength

Basic Considerations.—Compressive strength of the foundation rock can be an important factor in determining thickness requirements for a dam at its contact with the foundation. Where the foundation rock is nonhomogeneous, tests to obtain compressive strength values should be made for each type of rock in the loaded portion of the foundation.

A determination of tensile strength of the rock is seldom required because unhealed joints, shears, etc., cannot transmit tensile stress within the foundation.

Criteria.—A sufficient number of tests, as determined by the designer, should be made to obtain compressive strength values for each type of rock in the loaded part of the foundation.

LOADS

Factors to be considered as contributing to the loading combinations for an arch dam are: (1) reservoir and tailwater loads, (2) temperature, (3) internal hydrostatic pressure, (4) dead weight, (5) ice, (6) silt, and (7) earthquake. Such factors as dead weight and static water loads can be calculated accurately. Others such as earthquake, temperature, ice, silt, and internal hydrostatic pressures must be predicted on the basis of assumptions of varying reliability.

Reservoir and Tailwater Loads

Basic Considerations.—Reservoir and tailwater loads to be applied to the dam are obtained from reservoir operation studies and tailwater curves. These studies are based on operating and hydrologic data such as reservoir capacity, storage allocations, streamflow records, flood hydrographs, and reservoir releases for all purposes. A design reservoir can be derived from these operation studies which will reflect a normal high water surface, seasonal drawdowns, and the usual low water surface. Definitions of the water surface designations are:

(1) *Maximum water surface.*—The highest acceptable water surface elevation with all factors affecting the safety of the structure considered. Normally it is the highest water surface elevation resulting from a computed routing of the inflow design flood through the reservoir on the basis of established operating criteria. It is the top of surcharge capacity.

(2) *Top of exclusive flood control capacity.*—The reservoir water surface elevation at the top of the reservoir capacity allocated to exclusive use of regulating flood inflows to reduce damage downstream.

(3) *Maximum controllable water surface elevation.*—The highest reservoir water surface elevation at which gravity flows from the reservoir can be completely shut off.

(4) *Top of joint use capacity.*—The reservoir water surface elevation at the top of the reservoir capacity allocated to the joint uses of flood control and conservation purposes.

(5) *Top of active conservation capacity.*—The reservoir water surface elevation at the top of the capacity allocated to storage of water for conservation purposes only.

(6) *Top of inactive capacity.*—The reservoir water surface elevation below which the reservoir will not be evacuated under normal conditions.

(7) *Top of dead capacity.*—The lowest elevation in the reservoir from which water can be drawn by gravity.

(8) *Streambed at the dam axis.*—The elevation of the lowest point in the streambed at the axis of the dam prior to construction. This elevation normally defines the zero for area-capacity tables.

The normal design reservoir elevation is the "Top of Joint Use Capacity" if joint use capacity is included. If not, it is the "Top of Active Conservation Capacity."

The minimum design reservoir elevation is defined as the usual low-water surface as reflected in seasonal drawdowns. Unless the reservoir is drawn down to "Top of Inactive Capacity" at frequent intervals, the minimum design reservoir elevation will be higher than that level.

Maximum design reservoir elevation is the highest anticipated water surface elevation and usually occurs in conjunction with routing of the inflow design flood through the reservoir.

For computation of the reservoir and tailwater loads, water pressure is considered to vary directly with depth and to act equally in all directions.

Criteria.—Reservoir levels should be selected from reservoir operation studies for the loading combinations being analyzed.

The minimum tailwater level associated with each reservoir level should be used. Tailwater surface elevations should be obtained from tailwater curves associated with operating studies.

Water pressures should be computed as varying directly with depth.

Water loads are considered to be applied at, and to act normal to, the contact surfaces.

Temperature

Basic Considerations.—A concrete arch dam is subjected to temperature loads when a temperature change occurs in the concrete and the resulting volumetric change is restrained by the adjacent blocks and the abutments. The magnitudes of the

temperature loads are the differences between the closure temperature and concrete temperatures to be expected in the dam during operation [4].

The closure temperature of an arch dam is defined as the mean concrete temperature at the time the structure is assumed to act monolithically and arch action begins. Generally, a monolithic structure is obtained by grouting the vertical contraction joints or by backfilling the closure slot.

Natural cooling of the concrete will result in variations in closure temperatures, depending upon height and thickness of the dam, climatic conditions, and the construction schedule.

By artificially cooling the concrete with embedded temperature control systems, the closure temperature can be controlled to be uniform throughout the dam or varied over the height of the structure to achieve the desired stress distribution. The closure temperature or temperatures to be incorporated into the design should be determined from results of stress analyses and modified as necessary by practical considerations such as costs of temperature control measures, site conditions, and construction programs.

Concrete temperatures to be expected during operation of the dam are determined from studies which include the effects of ambient air temperatures, reservoir water temperatures, and solar radiation.

When the designer is making studies to determine concrete temperature loads and temperature gradients, varying weather conditions can be applied. Similarly, a widely fluctuating reservoir water surface will affect the concrete temperatures. In determining temperature loads, the following conditions and temperatures are used.

(1) *Mean air temperature.*—The average air temperature which is expected to occur at the site. These are normally obtained from the National Weather Service records of the mean monthly air temperatures and the mean daily maximum and minimum air temperatures.

(2) *Usual weather conditions.*—The combination of the daily air temperatures, a 1-week cycle representative of the cold (hot) periods associated with barometric pressure changes, and the mean monthly air temperatures. This condition will account for temperatures which are halfway between the mean monthly air temperatures and the minimum (maximum) recorded air temperatures at the site.

(3) *Extreme weather conditions.*—The combination of the daily air temperatures, a 2-week cycle representative of the cold (hot) periods associated with barometric pressure changes, and the mean monthly air temperatures. This condition will account for the minimum (maximum) recorded air temperatures at the site. This is an extreme condition and is seldom used.

(4) *Mean concrete temperatures.*—The average concrete temperatures between the upstream and downstream faces which will result from mean air temperatures, reservoir water temperatures associated with the design reservoir operation, and solar radiation.

(5) *Usual concrete temperatures.*—Same as above, except that *usual* weather conditions are applied.

(6) *Extreme concrete temperatures.*—Same as above, except that *extreme* weather conditions are applied.

Criteria.—For reconnaissance and feasibility designs, temperature studies that give the range of mean concrete temperatures are satisfactory. Final design studies should use such methods as Schmidt's [4], or finite element methods to determine concrete temperatures and temperature gradients between the faces of the dam as they vary throughout the year in different parts of the structure.

Internal Hydrostatic Pressures

Basic Considerations.—Hydrostatic pressures caused by reservoir and tailwater pressures occur within the dam and foundation in pores, joints, cracks, and seams. The distribution and magnitudes of these internal pressures can be modified by placing formed drains near the upstream face of the dam. When drains are used, the lateral spacing and distance from the upstream face will depend on the maximum reservoir depth and thickness of the dam. Drains are constructed roughly parallel to the vertically curved upstream face of recently designed Bureau arch dams which are curved in both vertical and horizontal directions.

Internal hydrostatic pressures reduce the compressive stresses acting within the concrete, thereby lowering the frictional shear resistances. Unlike gravity dams, which depend on shear resistance for stability, arch dams resist much of the applied load

by transferring it horizontally to the abutments by arch action.

The effects of any internal hydrostatic pressures in arch dams, therefore, will be distributed between both vertical and horizontal elements. A recent analysis of these effects on an arch dam of moderate height showed a stress change of approximately 5 percent of the allowable stress. The capability of analyzing the effects of internal hydrostatic pressure has not been incorporated as a regular part of the analysis because of the minor change in stress.

The internal pressure distribution through the foundation depends on depth of drains, grout curtain, rock porosity, jointing, faulting, and any other geologic features that may modify the flow. Determination of such pressure distributions can be made from flow nets computed by several methods including two- and three-dimensional physical models, two- and three-dimensional finite element models, and electric analogs.

Criteria.—Because of the small effect of internal hydrostatic pressure within an arch dam, it is not included in the design analyses. However, formed drains in the dam should be utilized, where appropriate, to minimize internal hydrostatic pressures. Foundation drainage and grouting should be included to reduce the internal hydrostatic pressures within the foundation.

Dead Load

Basic Considerations.—Dead load is the weight of concrete plus such appurtenances as gates and bridges. The construction and grouting sequence can affect the manner in which dead load is transmitted to the foundation. All dead load imposed on the structure prior to the grouting of contraction joints is assumed to be transmitted vertically to the foundation without any transfer of shear across the ungrouted joints. Dead load imposed after grouting of contraction joints is assumed to be distributed between vertical and horizontal elements in much the same manner as other loads.

Criteria.—The magnitude of dead load is considered as the weight of concrete plus appurtenances.

Effects of the construction and grouting sequence should be considered in determining the distribution of dead load.

Ice

Basic Considerations.—Ice pressures can produce a significant load against the face of a dam in

locations where winter temperatures are cold enough to cause a relatively thick ice cover. Ice pressure is created by thermal expansion of the ice and by wind drag. Pressures caused by thermal expansion of the ice depend on the temperature rise of the ice, thickness of the ice sheet, the coefficient of thermal expansion, the elastic modulus, and the strength of the ice. Wind drag depends on the size and shape of the exposed area, the roughness of the surface, and the direction and velocity of the wind. Ice pressures are generally considered to be a transitory loading. Many dams will be subjected to little, if any, ice pressure. The designer should decide, after consideration of the above factors, if an allowance for ice pressure is appropriate.

Criteria.—The method of Monfore and Taylor [8] may be used to analyze anticipated ice pressures if necessary basic data are available. An acceptable estimate of ice load to be expected on the face of a structure may be taken as 10,000 lbs/lin ft (146 kN/m) of contact between the ice and the dam for an assumed ice depth of 2 feet (0.6 meter) or more when basic data are not available to compute pressures.

Silt

Basic Considerations.—Not all dams will be subjected to silt pressures, and the designer should consider all available hydrologic data before deciding whether an allowance for silt pressure is necessary.

Criteria.—Horizontal pressure exerted by the saturated silt load is assumed to be equivalent to that of a fluid weighing 85 lbs/ft³ (1362 kg/m³). Vertical pressure exerted by saturated silt is determined as if silt were a soil having a wet density of 120 lbs/ft³ (1922 kg/m³), the magnitude of pressure varying directly with depth.

Earthquake

Basic Considerations.—Concrete arch dams are elastic structures which may be excited to resonance when subjected to seismic disturbances. Two steps are necessary to obtain loadings on an arch dam when such a disturbance occurs. The first step is to estimate magnitudes and locations of earthquakes to which the dam may be subjected and to determine the resulting rock motions at the site. The second step is to determine the response of the dam to these earthquakes by either the response spectrum or time-history methods.

Methods of determining a design earthquake which represents an operating basis event are presently under development. These methods will consider: (1) historical records to obtain frequency of occurrence versus magnitude, (2) useful life of the structure, and (3) a statistical approach to determine probable occurrence of different magnitudes of earthquakes during the life of the structure. When future developments produce such methods, suitable safety factors will be included in the criteria.

Maximum Credible Earthquake.—Most earthquakes are the result of crustal movements of the earth along faults. Geologic examinations of the area should be made to locate any faults, determine how recently activity has occurred, and estimate the probable length of the fault. Records of seismological activity in the area should also be studied to determine magnitude and location of any recorded earthquakes which may affect the site. Based on these geological and historical data, hypothetical earthquakes, usually having magnitudes greater than the historical events, are estimated for any active faults in the area. These earthquakes are considered the most severe associated with the faults and are assumed to occur at the points on those faults closest to the site. This defines the "Maximum Credible Earthquake" in terms of Richter magnitudes and distances to the site.

The field of earthquake engineering is currently the subject of much research and development. The following criteria represent the current concepts used by the Bureau to obtain earthquake loads for concrete arch dams.

Criteria.—The dam should be analyzed for the "Maximum Credible Earthquake."

(1) *Response spectrum.*—A response spectrum at the site should be determined for each "Maximum Credible Earthquake" by all three methods described in appendix D of reference [9]. The composite of the three spectra becomes the design response spectrum.

(2) *Time history.*—The required accelerograms may be produced by appropriate adjustment of existing or artificially generated accelerograms. The response spectrum computed from an adjusted accelerogram must correspond to the above-defined design response spectrum.

(3) *Structural response.*—The analytical methods used to compute natural frequencies,

mode shapes, and structural response are described in another Reclamation publication [1].

LOAD COMBINATIONS

Basic Considerations.—Arch dams should be designed for all appropriate load combinations using the proper safety factor for each. Combinations of transitory loads, each of which has only a remote probability of occurrence at any given time, have a negligible probability of simultaneous occurrence and should not be considered as an appropriate load combination. When large fluctuations of the water level and temperature may be expected, the design should give an acceptable balance of stresses for the different applicable load combinations.

Criteria.—An arch dam should be designed for the applicable load combinations using the safety factors prescribed under "Factors of Safety" in this monograph. The loading combinations to be considered are as follows:

(1) *Usual loading combinations.*—(a) Effects of minimum usual concrete temperatures and the most probable reservoir elevation occurring at that time with appropriate dead loads, tailwater, ice, and silt.

(b) Effects of maximum usual concrete temperatures and the most probable reservoir elevation occurring at that time with appropriate dead loads, tailwater, and silt.

(c) Normal design reservoir elevation and the effects of usual concrete temperatures occurring at that time with appropriate dead loads, tailwater, ice, and silt.

(d) Minimum design reservoir elevation and the effects of concrete temperatures occurring at that time with appropriate dead loads, tailwater, ice, and silt.

(2) *Unusual loading combinations.*—Maximum design reservoir elevation and the effects of mean concrete temperatures occurring at that time with appropriate dead loads, tailwater, and silt.

(3) *Extreme loading combinations.*—Any of the usual loading combinations plus effects of the "Maximum Credible Earthquake."

(4) *Other loadings and investigations.*—(a) Any of the above loadings plus hydrostatic pressures within the foundation for foundation stability studies.

(b) Dead load.

(c) Any of the above loadings revised to reflect

the chronological application of loadings to include the effects of construction and grouting sequences if the contraction joints are to be grouted in stages.

(d) Any other loading combination which, in the designer's opinion, should be analyzed for a particular dam.

CONFIGURATION OF DAM AND FOUNDATION

Basic Considerations.—The shape of an arch dam and the configuration of its foundation contacts are extremely important in providing stability and favorable stress conditions. Proper curvatures of the dam in both horizontal and vertical directions are desirable to obtain acceptable stresses and an economic design. The greatest degree of horizontal curvature consistent with the limitation on maximum central angle at the crest and other requirements for the site will generally provide the most effective arch action for circular horizontal arches in relatively narrow sites. When the economy of the structure is considered, the maximum practicable angle is between 90° and 110° for the top arch. Similarly, effective arch action can be achieved by using polycentered horizontal arches in the wider sites.

Added thickness at the abutments of the dam can be used to control and distribute stresses transmitted to the foundation by the use of abutment pads, fillets, or variable thickness arches.

The angle of incidence of a tangent to the intrados with contours for the competent rock should be large enough to provide adequate foundation rock cover in the direction of the resultant force. The contact of the dam with the canyon should be smooth without any abrupt changes either in configuration of arch abutments or in slope along the abutment contact profile, to minimize stress concentrations.

The following criteria are generally applicable for the design of most arch dams. Special studies for unusual cases, however, may indicate that increased economy and adequate safety can be achieved by deviating from the criteria.

Criteria.—The maximum degree of horizontal curvature compatible with site conditions should be used. The maximum central angle of the crest, however, should be between 90° and 110° . The angle of incidence of a tangent to the intrados with contours for competent rock should not be less than 30° .

The arches should have abutments radial to the axis center where feasible. If this results in excessive excavation on the upstream face, full radial abutments at the top of the dam should be merged without abrupt changes into half radial abutments, as shown on figure 2(a). If radial abutments would result in excessive excavation on the downstream face, the abutment may be greater than radial, as shown on figure 2(b). These nonradial abutments should be merged smoothly into radial abutments.

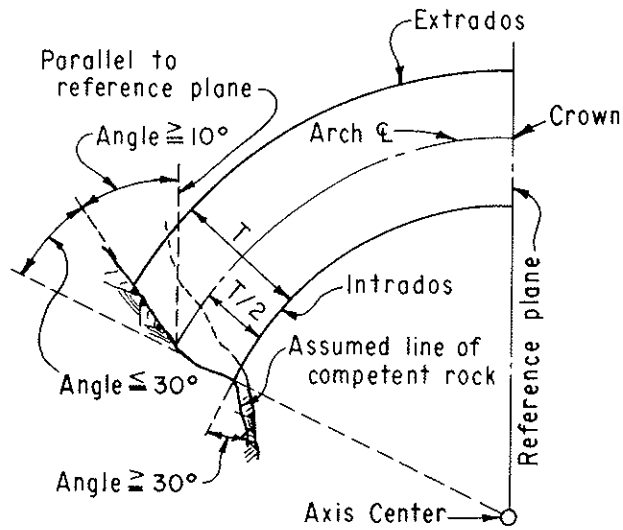
CRACKING

Basic Considerations.—Horizontal cracking should be assumed to occur in an arch dam wherever vertical tensile stresses resulting from dynamic response of the structure to an earthquake exceed the tensile strength of the concrete. Cracking would be initiated at the point of maximum tensile stress first. The cracks should be assumed to extend to the point of zero stress within the dam. Although internal hydrostatic pressures may exist within the dam, pressure in the crack during earthquake is assumed to be zero. This assumption accounts for the rapidly cycling nature of opening and closing of the cracks and the inability of internal hydrostatic pressure to develop rapidly. The vertical contraction joints are assumed to open whenever horizontal tensile stresses are indicated. If stresses indicate their possible existence, effects of both horizontal cracks and opening of the vertical joints during an earthquake should be evaluated. Horizontal or peripheral formed joints may be used to eliminate tensile stresses in a manner similar to the stress relief during cracking.

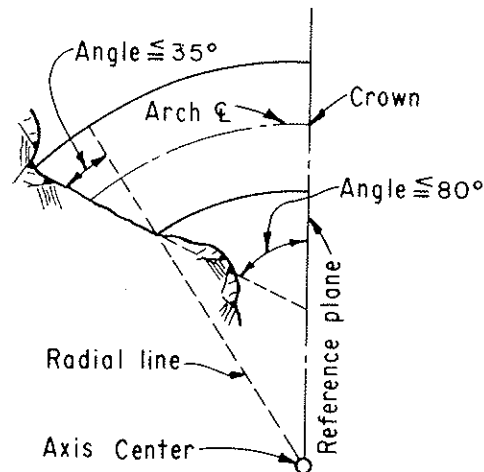
Methods of determining the depth of crack, resulting stresses, etc., are discussed in other Bureau of Reclamation publications [1].

Criteria.—Wherever computed tensions exceed the tensile strength of the concrete, cracking should be assumed to exist. The depth of crack extends to the point of zero stress within the section.

After the change in static flexibility caused by the cracking and opening of contraction joints or other formed joints has been included, another analysis should be made. Any changes in depth of cracking or openings that result should be incorporated and the procedure repeated until stresses computed at the ends of the cracks or opened joints are zero and further changes in depth are not indicated.



(a) --- Half Radial Arch Abutment



(b) --- Greater Than Radial Abutment

Figure 2.—Arch abutment shaping criteria.

FACTORS OF SAFETY

Basic Considerations.—All loads to be used in the design should be chosen to represent, as nearly as can be determined, the actual loads that will act on the structure during operation, in accordance with the criteria under "Load Combinations." Methods of determining load-resisting capacity of the dam should be the most accurate available. All uncertainties regarding loads or load-carrying capacity must be resolved as far as practicable by field or laboratory tests, thorough exploration and inspection of the foundation, good concrete control, and

good construction practices. On this basis, the factor of safety will be as accurate an evaluation as possible of the capacity of the structure to resist applied loads. All safety factors listed are minimum values.

Like other important structures, dams should be frequently inspected. Particularly where uncertainties exist regarding such factors as loads, resisting capacity, or characteristics of the foundation, it is expected that adequate observations and measurements will be made of the structural behavior of the dam and its foundation to assure that the structure is at all times functioning as designed.

Although somewhat lower safety factors may be permitted for limited local areas, overall safety factors for the dam and its foundation after beneficiation should meet the requirements for the loading combination being analyzed.

For other loading combinations where the safety factors are not specified, the designer is responsible for the selection of safety factors consistent with those for the loading combination categories previously discussed. Somewhat higher safety factors should be used for foundation studies because of the greater amount of uncertainty involved in assessing foundation load-resisting capacity.

Safety factors for the dam are based on analyses using the "Trial Load Method" or its computerized version, the Arch Dam Stress Analysis System (ADSAS) [1]. Those for the foundation sliding stability are based on an assumption of uniform shear stress distributed on the plane being analyzed.

Criteria.—(1) *Compressive stress.*—The maximum allowable compressive stress for concrete in an arch dam subjected to any of the "Usual Loading Combinations" should be equal to the specified compressive strength divided by a safety factor of 3.0. However, in no case should the allowable stress for "Usual Loading Combinations" exceed 1,500 lbs/in² (10.3 MPa).

The maximum allowable compressive stress for the "Unusual Loading Combinations" should be equal to the specified compressive strength divided by a safety factor of 2.0, and in no case should this value exceed 2,250 lbs/in² (15.5 MPa).

The maximum allowable compressive stress for the "Extreme Loading Combinations" should be determined in the same way, using a safety factor greater than 1.0.

Safety factors of 4.0, 2.7, and 1.3 should be applied to the foundation compressive strength

in determining the allowable compression in the foundation for "Usual," "Unusual," and "Extreme Loading Combinations," respectively.

(2) *Tensile stress*.—Whenever practicable, tensile stresses should be avoided by redesign of the dam. However, limited amounts of tensile stress may be permitted in localized areas for the "Usual Loading Combinations" at the discretion of the designer. Under no circumstances should this tensile stress exceed 150 lbs/in² (1.03 MPa) and for the "Unusual Loading Combinations" should be no greater than 225 lbs/in² (1.55 MPa).

For the low-reservoir—high-temperature loading combination or for dead load during concrete placement when tensile stresses may occur on the downstream face, a somewhat higher tensile stress may be permitted in localized areas that are under compression during full reservoir loading. An allowable tensile stress equal to the tensile strength of concrete at the lift surface may be permitted at the discretion of the designer. The point of application of the resultant dead load force must also remain within the base of the vertical section to maintain stability during construction.

For the "Extreme Loading Combinations," which includes the "Maximum Credible Earthquake," the concrete should be assumed to crack when the tensile strength is exceeded. The cracks are assumed to propagate to the point of zero stress as required under "Cracking" in this monograph. The structure may be deemed safe for the "Extreme Loading Combinations" if, after cracking effects have been included, the stresses do not exceed the strength and the stability of the structure is maintained.

(3) *Shear stress and sliding stability*.—The maximum allowable average shearing stress on any plane within the dam should be determined by dividing the shear strength by 3.0 for the "Usual Loading Combinations," and by 2.0 for the "Unusual Loading Combinations." A safety factor greater than 1.0 is required for the "Extreme Loading Combinations."

Where cracking for the "Extreme Loading Combinations" is included, the remaining por-

tion of the uncracked section should be checked for stability against sliding. The shear-friction factor of safety, which is a measure of stability against sliding, should be used to check the reliability of the remainder of the partially cracked section. The expression for the shear-friction factor of safety (Q) is the ratio of resisting to driving forces, as follows:

$$Q = \frac{CA + (\Sigma N + \Sigma U) \tan \phi}{\Sigma V}$$

where:

- C = unit cohesion
- A = uncracked area
- ΣN = summation of normal forces
- ΣU = summation of uplift forces
- $\tan \phi$ = coefficient of internal friction
- ΣV = summation of shear forces

All parameters must be specified in consistent units. Uplift is negative according to the sign convention given in reference [1].

To assure safety against sliding during this extreme loading, the shear-friction factor should have a value greater than 1.0. The same equation can be used to determine the sliding stability at the concrete-rock contact or at any plane within the dam for the other loading combinations instead of checking the allowable shear stress. Adequate shear-friction factors of safety are 3.0 for the "Usual," 2.0 for the "Unusual," and greater than 1.0 for the "Extreme Loading Combinations."

Methods of determining stability against sliding within the foundation are discussed in chapter 4 of reference [1].

The factor of safety against sliding of the entire structure along planes of possible weakness within the foundation should be no less than 4.0 for the "Usual," 2.7 for the "Unusual," and 1.3 for the "Extreme Loading Combinations." If the computed safety factor is less than required, foundation treatment can be included to increase the safety factor to the required value.

Part II Gravity Dams

Gravity Dams

CONCRETE PROPERTIES—STATIC

Strength

Basic Considerations.—A gravity dam should be constructed of concrete that will meet the design criteria for strength, durability, permeability, and other required properties. Because of the sustained loading generally associated with them, the concrete properties used for the analyses of static loading conditions should include the effect of creep. Properties of concrete vary with age, the type of cement, aggregates, and other ingredients as well as their proportions in the mix [3]. Since different concretes gain strength at different rates, measurements must be made of specimens of sufficient age to permit evaluation of ultimate strengths.

Although the concrete mix is usually designed for only compressive strength, appropriate tests should be made to determine the tensile and shear strength values.

Elastic Properties

Basic Considerations.—Poisson's ratio, the sustained modulus of elasticity of the concrete, and the latter's ratio to the deformation modulus of the foundation have significant effects on stress distribu-

tion in the structure. Values of the modulus of elasticity, although not directly proportional to concrete strength, do follow the same trend, with the higher strength concretes having a higher value for modulus of elasticity. As with the strength properties, the elastic modulus is influenced by mix proportions, cement, aggregate, admixtures, and age. The deformation that occurs immediately with application of load depends on the instantaneous elastic modulus. The increase in deformation which occurs over a period of time with a constant load is the result of creep or plastic flow in the concrete. The effects of creep are generally accounted for by determining a sustained modulus of elasticity of the concrete for use in the analyses of static loadings.

Instantaneous moduli of elasticity and Poisson's ratios should be determined for the different ages of concrete when the cylinders are initially loaded. The sustained modulus of elasticity should be determined from these cylinders after specific periods of time under constant sustained load. These periods of loading are often 365 and 730 days. The cylinders to be tested should be of the same size and cured in the same manner as those used for the compressive strength tests. The values of instantaneous modulus of elasticity, Poisson's ratio, and sustained modulus

of elasticity used in the analyses should be the average of all test cylinder values.

Thermal Properties

Basic Considerations.—The effects of temperature change in gravity dams are not as important in the design as those in arch dams. However, during construction, the temperature change of the concrete in the dam should be controlled to avoid undesirable cracking. Thermal properties necessary for the evaluation of temperature changes are the coefficient of thermal expansion, thermal conductivity, specific heat, and diffusivity [4].

The coefficient of thermal expansion is the length change per unit length for 1 degree temperature change. Thermal conductivity is the rate of heat conduction through a unit thickness over a unit area of the material subjected to a unit temperature difference between faces. The specific heat is defined as the amount of heat required to raise the temperature of a unit mass of the material 1 degree. Diffusivity of concrete is an index of the facility with which concrete will undergo temperature change. The diffusivity is calculated from the values of specific heat, thermal conductivity, and density.

Criteria.—Appropriate laboratory tests should be made of the design mix to determine all concrete properties.

CONCRETE PROPERTIES—DYNAMIC

Strength

The Reclamation laboratory is presently testing for the strength of concrete when the concrete is subjected to dynamic loading. However, no data are yet available to indicate what the strength characteristics are under dynamic loading.

Elastic Properties

Until dynamic modulus information is available, the instantaneous modulus of elasticity determined for concrete specimens at the time of initial loading should be the value used for analyses of dynamic effects.

AVERAGE PROPERTIES

Basic Considerations.—Necessary values of concrete properties may be estimated from published data for preliminary studies until laboratory test data are available. Until long-term tests are made to

determine the effects of creep, the sustained modulus of elasticity should be taken as 60 to 70 percent of the laboratory value for the instantaneous modulus of elasticity.

Criteria.—If no tests or published data are available, the following average values for concrete properties may be used for preliminary designs until test data are available for better results.

Compressive strength—3,000 to 5,000 lbs/in²
(20.7 to 34.5 MPa)

Tensile strength—5 to 6 percent of the compressive strength

Shear strength:

Cohesion—about 10 percent of the compressive strength

Coefficient of internal friction—1.0

Poisson's ratio—0.2

Instantaneous modulus of elasticity— 5.0×10^6 lbs/in² (34.5 GPa)

Sustained modulus of elasticity— 3.0×10^6 lbs/in² (20.7 GPa)

Coefficient of thermal expansion— $5.0 \times 10^{-6}/^{\circ}\text{F}$
($9.0 \times 10^{-6}/^{\circ}\text{C}$)

Unit weight—150 lbs/ft³ (2402.8 kg/m³)

FOUNDATION PROPERTIES

Deformation Modulus

Basic Considerations.—Foundation deformations caused by loads from the dam affect the stress distributions within the dam. Conversely, response of the dam to external loading and foundation deformability determines the stresses within the foundation. Proper evaluation of the dam and foundation interaction requires as accurate a determination of foundation deformation characteristics as possible.

Although the dam is considered to be homogeneous, elastic, and isotropic, its foundation is generally heterogeneous, inelastic, and anisotropic. These characteristics of the foundation have significant effects on the deformation moduli of the foundation. The analysis of a gravity dam should include the effective deformation modulus and its variation over the entire contact area of the dam with the foundation.

The deformation modulus is defined as the ratio of applied stress to elastic strain plus inelastic strain and should be determined for each foundation material. The effective deformation modulus is a composite of deformation moduli for all materials within a particular segment of the foundation.

Foundation investigations should provide information related to or giving deformation moduli and elastic moduli. The information includes elastic modulus of drill core specimens, elastic modulus and deformation modulus from in situ jacking tests, deformation modulus of fault or shear zone material, and logs of the jointing that occurs in recovered drill core. Information on the variation of materials and their prevalence at different locations along the foundation is provided by the logs of drill holes and by tunnels in the foundation. Good compositional description of the zone tested for deformation modulus and adequate geologic logging of the drill cores permit extrapolation of results to untested zones of similar material.

Criteria.—The following foundation data should be obtained for the analysis of a gravity dam:

- The deformation modulus of each type of material within the loaded area of the foundation.
- The effects of joints, shears, and faults obtained by direct (testing) or indirect (reduction factor) methods [10].
- An effective deformation modulus, as determined when more than one type of material is present in a foundation.
- The effective deformation moduli, as determined at enough locations along the foundation contact to provide adequate definition of the variation in deformability and to permit extrapolation to untested areas when necessary.

Shear Strength

Basic Considerations.—Resistance to shear within the foundation and between the dam and its foundation depends upon the cohesion and internal friction inherent in the foundation materials and in the bond between concrete and rock at the contact with the dam. These properties are determined from laboratory and in situ tests. The results of laboratory triaxial and direct shear tests, as well as in situ shear tests, are generally reported in the form of the Coulomb equation:

$$R = CA + N \tan \phi, \text{ or}$$

(Shear resistance) = (unit cohesion times area)
+ (effective normal force times
coefficient of internal friction)

which defines a linear relationship between shear resistance and normal load. The value of shear

resistance obtained as above should be limited to use for the range of normal loads used for the tests.

Although this assumption of linearity is usually realistic for the shear resistance of intact rock over the range of normal loads tested, a curve of shear resistance versus normal load should be used for materials other than intact rock. The shear resistance versus normal load relationship is determined from a number of tests at different normal loads. The individual tests give the relationship of shear resistance to displacement for a particular normal load. The results of these individual tests are used to obtain a shear resistance versus normal load curve, as shown on figure 3. The displacement used to determine the shear resistance is the maximum displacement that can be allowed on the possible sliding plane without causing unacceptable stress concentrations within the dam. Since specimens tested in the laboratory or in situ are small compared to the foundation, the scale effect should be carefully considered in determining the values of shear resistance to be used.

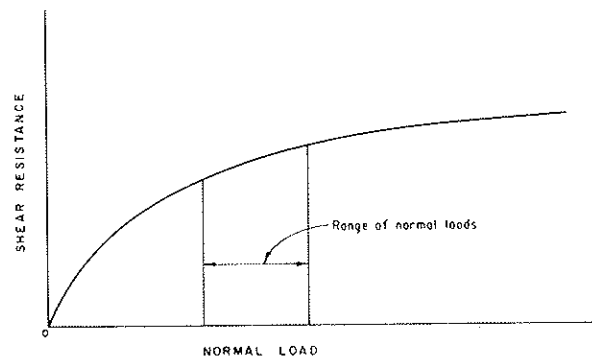


Figure 3.—Shear resistance on an existing joint in rock foundation of a gravity dam.

When a foundation is nonhomogeneous, the possible sliding surface may consist of several different materials, some intact and some fractured. Intact rock reaches its maximum break bond resistance with less deformation than is necessary for fractured materials to develop their maximum frictional resistances. Therefore, the shear resistance developed by each fractured material depends upon the displacement of the intact rock part of the surface. If the intact rock shears, the shear resistance of the entire plane is equal to the combined sliding frictional resistance for all materials along the plane.

Criteria.—An adequate number of tests, as determined by the designer, should be made to obtain a shear resistance versus normal load relationship for each material along the possible sliding planes. The value of shear resistance recorded during tests should be measured at displacements which correspond to those expected to occur along the in situ potential sliding planes.

When the foundation along the plane of potential sliding is nonhomogeneous, the total shear resistance is the summation of shear resistances of all the materials along the plane.

Pore Pressure and Permeability

Basic Considerations.—Analysis of a dam foundation requires a knowledge of the hydrostatic pressure distribution in the foundation. Permeability is controlled by the characteristics of the rock type, the jointing systems, the shears and fissures, fault zones, and, at some damsites, by solution cavities in the rock. The exit gradient for shear zone materials that surface near the downstream toe of the dam should also be determined to check against the possibility of piping [2].

Laboratory values for permeability of sample specimens are applicable only to the portion or portions of the foundation that they represent.

Permeability of the aforementioned geologic features can best be determined by in situ testing. The permeabilities obtained are used in the determination of pore pressures for analyses of stresses, stability, and piping. Such a determination may be made by several methods including two- and three-dimensional physical models, two- and three-dimensional finite element models, and electric analogs.

If foundation grouting and drainage or other treatment are to be used, their effects on the pore pressures should be included.

Criteria.—A sufficient number of tests, as determined by the designer, should be made to determine the permeability of the foundation rock, joints, fissures, fault zones, and solution cavities [6,7].

An adequate method of analysis should be used to determine pore pressures within the foundation. The effects of any grouting, drainage, and other foundation treatment should be included.

Treatment

Basic Considerations.—Foundation treatment is used to correct deficiencies and improve physical properties by grouting, drainage, excavation of

inadequate materials, reinforcement and backfill with concrete. Some reasons for foundation treatment are: (1) improvement of deformation moduli, (2) prevention of sliding of foundation blocks, (3) prevention of relative displacement of foundation blocks, (4) prevention of piping and reduction of pore pressures, and (5) provision of an artificial foundation in the absence of adequate materials.

Regardless of the reason for the foundation treatment, its effects on the other foundation properties should be considered in the analyses.

Criteria.—Effects of treatment on foundation properties should be considered.

Compressive and Tensile Strength

Basic Considerations.—Compressive strength of the foundation rock can be an important factor in determining thickness requirements for a dam at its contact with the foundation. Where the foundation rock is nonhomogeneous, tests to obtain compressive strength values should be made for each type of rock in the loaded portion of the foundation.

A determination of tensile strength of the rock is seldom required because unhealed joints, shears, etc., cannot transmit tensile stress within the foundation.

Criteria.—A sufficient number of tests, as determined by the designer, should be made to obtain compressive strength values for each type of rock in the loaded part of the foundation.

LOADS

Factors to be considered as contributing to the loading combinations for a gravity dam are: (1) reservoir and tailwater loads, (2) temperature, (3) internal hydrostatic pressure, (4) dead weight, (5) ice, (6) silt, and (7) earthquake. Such factors as dead weight and static water loads can be calculated accurately. Others such as earthquake, temperature, ice, silt, and internal hydrostatic pressure must be predicted on the basis of assumptions of varying reliability.

Reservoir and Tailwater Loads

Basic Considerations.—Reservoir and tailwater loads to be applied to the dam are obtained from reservoir operation studies and tailwater curves. These studies are based on operating and hydrologic data such as reservoir capacity, storage allocation, streamflow records, flood hydrographs, and reser-

voir releases for all purposes. A design reservoir can be derived from these operation studies which will reflect a normal high water surface, seasonal drawdowns, and the usual low water surface. Definitions of the water surface designations are:

(1) *Maximum water surface.*—The highest acceptable water surface elevation with all factors affecting the safety of the structure considered. Normally, it is the highest water surface elevation resulting from a computed routing of the inflow design flood through the reservoir on the basis of established operating criteria. It is the top of surcharge capacity.

(2) *Top of exclusive flood control capacity.*—The reservoir water surface elevation at the top of the reservoir capacity allocated to exclusive use of regulating flood inflows to reduce damage downstream.

(3) *Maximum controllable water surface elevation.*—The highest reservoir water surface elevation at which gravity flows from the reservoir can be completely shut off.

(4) *Top of joint use capacity.*—The reservoir water surface elevation at the top of the reservoir capacity allocated to joint uses of flood control and conservation purposes.

(5) *Top of active conservation capacity.*—The reservoir water surface elevation at the top of the capacity allocated to storage of water for conservation purposes only.

(6) *Top of inactive capacity.*—The reservoir water surface elevation below which the reservoir will not be evacuated under normal conditions.

(7) *Top of dead capacity.*—The lowest elevation in the reservoir from which water can be drawn by gravity.

(8) *Streambed at the dam axis.*—The elevation of the lowest point in the streambed at the axis of the dam prior to construction. This elevation normally defines the zero for area-capacity tables.

The normal design reservoir elevation is the "Top of Joint Use Capacity," if joint use capacity is included. If not, it is the "Top of Active Conservation Capacity."

The minimum design reservoir elevation is defined as the usual low water surface as reflected in seasonal drawdowns. Unless the reservoir is drawn down to "Top of Inactive Capacity" at frequent

intervals, the minimum design reservoir elevation will be higher than that level.

Maximum design reservoir elevation is the highest anticipated water surface elevation and usually occurs in conjunction with routing of the inflow design flood through the reservoir.

For computation of the reservoir and tailwater loads, water pressure is considered to vary directly with depth and to act equally in all directions.

Criteria.—Reservoir levels should be selected from reservoir operation studies for the loading combinations being analyzed.

The minimum tailwater level associated with each reservoir level should be used. Tailwater surface elevations should be obtained from tailwater curves associated with operating studies. Water pressures should be computed as varying directly with depth.

Water loads are considered to be applied at, and act normal to, the contact surfaces.

Temperature

Basic Considerations.—Volumetric increases caused by temperature rise will transfer load across transverse contraction joints if the joints are grouted. Horizontal thrusts which are caused by volumetric changes as temperature increases will result in a transfer of load across grouted contraction joints, increasing twist effects and the loading of the abutments as discussed in another Bureau publication [2]. UngROUTED contraction joints are assumed to offer no restraint on volumetric increase caused by temperature rise and no associated transfer of load, providing the mean concrete temperatures remain below the closure temperature.

When the designer is making studies to determine concrete temperature loads, varying weather conditions can be applied. Similarly, a widely fluctuating reservoir water surface will affect the concrete temperatures. In determining temperature loads, the following conditions and temperatures are used:

(1) *Usual weather conditions.*—The combination of the daily air temperatures, a 1-week cycle representative of the cold (hot) periods associated with barometric pressure changes, and the mean monthly air temperatures. This condition will account for temperatures which are halfway between the mean monthly air temperatures and the minimum (maximum) recorded air temperatures at the site.

(2) *Usual concrete temperatures.*—The average concrete temperatures between the up-

stream and downstream faces which will result from usual air temperatures, reservoir water temperatures associated with the design reservoir operation, and solar radiation [4].

Criteria.—The effects of temperature change should be investigated if joints are to be grouted and when the operating temperatures are above the closure temperature if joints are to be ungrouted.

Internal Hydrostatic Pressures

Basic Considerations.—Hydrostatic pressures caused by reservoir water and tailwater occur within the dam and foundation in pores, cracks, joints, and seams. The distribution of internal hydrostatic pressures along a horizontal section through the dam is assumed to vary linearly from full reservoir pressure at the upstream face to zero or tailwater pressure at the downstream face. When formed drains are included in the dam, the internal pressure distribution should be modified to reflect effects of size, location, and spacing of the drains.

The internal pressure distribution through the foundation depends on depth of drains, grout curtain, rock porosity, jointing, faulting, and any other geologic features that may modify the flow. Determination of such pressure distributions can be made from flow nets computed by several methods, including two- and three-dimensional physical models, two- and three-dimensional finite element models, and electric analogs.

The effect of internal hydrostatic pressure acts to reduce the vertical compressive stresses in the concrete on a horizontal section through the dam or at its base, and is referred to as uplift. Figure 4 illustrates actual measured uplift pressures at the concrete-rock contact as compared with design assumptions for Shasta Dam.

Laboratory tests indicate that for practical purposes pore pressures act over 100 percent of the area of any section through the concrete. Electric analog studies and recent finite element analyses indicate that location of the line of drains at a distance from the upstream face of 5 percent of the maximum reservoir depth at the dam is desirable. A lateral spacing of twice that distance will reduce the average pore pressure at the line of drains to tailwater pressure plus one-third the differential between tailwater and headwater pressures. The values are based on the assumption that the lowest elevation in the drainage gallery is at or below tailwater level or that pumping of the drains will be

a part of the operating criteria. If the gallery is above tailwater elevation, the pressure at the line of drains should be determined as though the tailwater level is equal to the gallery elevation. In no case should these pressures exceed those computed for the dam without drains. Internal pressures are assumed to be unaffected by earthquake accelerations because of their transitory nature.

Criteria.—For preliminary design purposes, uplift pressure distribution within a gravity dam, within the foundation, and at their contact are assumed to have an intensity at the line of drains equal to the tailwater pressure plus one-third the differential between headwater and tailwater pressures. The pressure gradient is then extended linearly to headwater and tailwater levels, respectively. If there is no tailwater, a similar pressure diagram is determined using zero instead of tailwater pressure. In all cases, pore pressures are assumed to act over 100 percent of the area.

For the final design, determination of internal pressures within the dam should be based on location and spacing of drains. Pressures in the foundation rock or at its contact with the dam should be determined based on geologic structures in the rock as well as on the location, depth, and spacing of drains as discussed in the criteria for foundations.

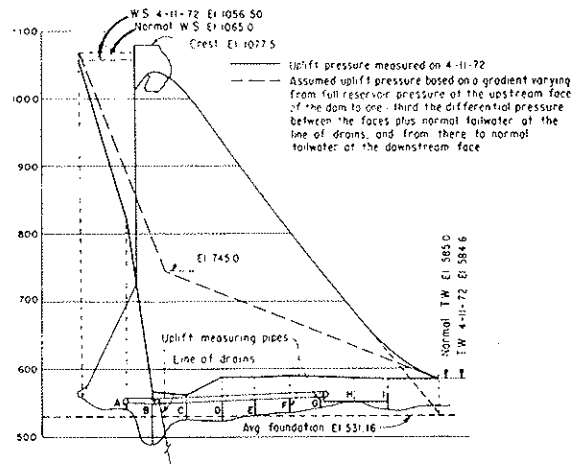


Figure 4.—Measured and computed uplift pressures for Shasta Dam.

Dead Load

Basic Considerations.—Dead load is the weight of concrete plus such appurtenances as gates and bridges. Dead loads are assumed to be transmitted vertically to the foundation without transfer of shear between adjacent blocks.

Criteria.—The magnitude of a dead load is considered as the weight of concrete plus appurtenances.

Ice

Basic Considerations.—Ice pressures can produce a significant load against the face of a dam in locations where winter temperatures are cold enough to cause relatively thick ice cover. Ice pressure is created by thermal expansion of the ice and by wind drag. Pressures caused by thermal expansion of the ice depend on the temperature rise of the ice, thickness of the ice sheet, the coefficient of thermal expansion, the elastic modulus, and the strength of the ice. Wind drag depends on the size and shape of the exposed area, the roughness of the surface, and the direction and velocity of the wind. Ice pressures are generally considered to be a transitory loading. Many dams will be subjected to little, if any, ice pressure. The designer should decide, after consideration of the above factors, if an allowance for ice pressure is appropriate.

Criteria.—The method of Monfore and Taylor [8] may be used to analyze anticipated ice pressures if necessary basic data are available. An acceptable estimate of ice load to be expected on the face of a structure may be taken as 10,000 lbs/lin ft (146 kN/m) of contact between the ice and the dam for an assumed ice depth of 2 feet (0.6 meter) or more when basic data are not available to compute pressures.

Silt

Basic Considerations.—Not all dams will be subjected to silt pressure, and the designer should consider all available hydrologic data before deciding whether an allowance for silt pressure is necessary.

Criteria.—Horizontal pressure exerted by the saturated silt load is assumed to be equivalent to that of a fluid weighing 85 lbs/ft³ (1362 kg/m³).

Vertical pressure exerted by saturated silt is determined as if silt were a soil having a wet density of 120 lbs/ft³ (1922 kg/m³), the magnitude of pressure varying directly with depth.

Earthquake

Basic Considerations.—Concrete gravity dams are elastic structures which may be excited to resonance when subjected to seismic disturbances. Two steps are necessary to obtain loadings on a

gravity dam caused by such a disturbance. The first step is to estimate magnitude and locations of earthquakes to which the dam may be subjected and determine the resulting rock motions at the site. The second step is to determine the response of the dam to these earthquakes by either the response spectrum or time-history methods.

Methods of determining a design earthquake which represents an operating basis event are presently under development. These methods should consider: (1) historical records to obtain frequency of occurrence versus magnitude, (2) useful life of the structure, and (3) a statistical approach to determine probable occurrence of earthquakes of different magnitudes during the life of the structure. When future developments produce such methods, suitable safety factors will be included in the criteria.

Maximum Credible Earthquake.—Most earthquakes are the result of crustal movements of the earth along faults. Geologic examinations of the area should be made to locate any faults, determine how recently activity has occurred, and estimate the probable length of the fault. Records of seismological activity in the area should also be studied to determine magnitude and location of any recorded earthquakes that may affect the site. Based on these geological and historical data, hypothetical earthquakes, usually having magnitudes greater than the historical events, are estimated for any active faults in the area. These earthquakes are considered to be the most severe associated with the faults and are assumed to occur at the points on those faults closest to the site. This defines the "Maximum Credible Earthquake" in terms of Richter magnitudes and distances to the site.

Earthquake engineering is currently the subject of much research and development. The following criteria represent the current concepts used by the Bureau to obtain earthquake loads for concrete gravity dams.

Criteria.—The dam should be analyzed for the "Maximum Credible Earthquake."

(1) *Response spectrum.*—A response spectrum at the site should be determined for each "Maximum Credible Earthquake" by all three methods described in appendix D of reference [9]. The composite of the three spectra becomes the design response spectrum.

(2) *Time history*.—The required accelerograms may be produced by appropriate adjustments of existing or artificially generated accelerograms. The response spectrum computed from an adjusted accelerogram must correspond to the above-defined design response spectrum.

(3) *Structural response*.—The analytical methods used to compute natural frequencies, mode shapes, and structural response are described in another publication [2].

LOAD COMBINATIONS

Basic Considerations.—Gravity dams should be designed for all appropriate load combinations, using the proper safety factor for each. Combinations of transitory loads, each of which has only a remote probability of occurrence at any given time, have negligible probability of simultaneous occurrence and should not be considered as an appropriate load combination. Temperature loadings should be included when applicable, as previously discussed in this monograph.

Criteria.—Gravity dams should be designed for the loading combinations which follow using the safety factors subsequently prescribed under "Factors of Safety" in this monograph.

(1) *Usual loading combinations*.—Normal design reservoir elevation with appropriate dead loads, uplift, silt, ice, and tailwater. If temperature loads are applicable, use minimum usual temperatures occurring at that time.

(2) *Unusual loading combinations*.—Maximum design reservoir elevation with appropriate dead loads, silt, tailwater, uplift, and minimum usual temperatures occurring at that time, if applicable.

(3) *Extreme loading combinations*.—The usual loading plus effects of the "Maximum Credible Earthquake."

(4) *Other loadings and investigations*.—(a) The usual or unusual loading combination with drains inoperative.

(b) Dead load.

(c) Any other loading combination which, in the designer's opinion, should be analyzed for a particular dam.

CONFIGURATION OF DAM AND FOUNDATION

Basic Considerations.—The shape of a gravity dam, its thickness at the contact with the foundation, and the slope of the concrete-rock contact are

important factors providing stability for the structure. Transversely, the foundation contact should be either horizontal or sloping upward toward the downstream face. Longitudinally, the profile should vary smoothly without abrupt changes to minimize stress concentrations.

Abrupt changes of slope on either face of the dam can cause unacceptable stress concentrations and should be avoided whenever possible.

Although a vertical downstream face near the top of the dam to provide a crest thickness for access is usually acceptable, the point of intersection with the sloping downstream face should be carefully checked for possible stress concentrations. Minimizing the mass near the top of the dam is beneficial in reducing the effects of earthquake.

Criteria.—The foundation contact should be either horizontal or sloping upward toward the downstream face in a direction normal to the axis. In addition, the foundation contact should vary smoothly along the axis profile of the dam without any abrupt changes.

Abrupt changes in slope on either face should be avoided where unacceptable stress concentrations may occur.

The minimum crest thickness consistent with other requirements should be used to reduce earthquake effects.

CRACKING

Basic Considerations.—Horizontal cracking should be assumed to occur in a gravity dam wherever the vertical normal stress ($\bar{\sigma}_1$ on figure 5(b)) for dynamic response to an earthquake does not meet minimum safety factor requirements. The depth of crack is assumed to extend along a horizontal section to the point where compressive stress computed without uplift and the internal hydrostatic pressure are equal.

When the analysis for the extreme loading combination of normal reservoir plus effects of the "Maximum Credible Earthquake" indicates that cracking should occur, uplift pressures within the crack are assumed to be zero. This assumption takes into account the rapidly cycling nature of the opening and closing of the crack and the inability of internal hydrostatic pressure to develop rapidly. Determination of the depth of crack becomes an iterative process in which the remaining uplift depends on the depth of crack and the depth of crack depends partially on the uplift acting in the uncracked portion. If the compressive stress at the

upstream face is less than the uplift (internal hydrostatic) pressure minus the tensile strength of the concrete, the pressure diagram should be revised as on figure 5(d). For the initial determination of cracking effects, an assumption of crack depth equal to one-half the thickness can be used. Uplift effects in the uncracked portion can then be determined and the depth of crack computed. This cycle of determining uplift and crack depth changes should be repeated until a satisfactory degree of accuracy has been obtained. Uplift pressure ordinates at the end of the crack are obtained by interpolation using an uplift pressure diagram, as shown on figure 5(c) and discussed under "Internal Hydrostatic Pressures" in this monograph.

For cracking computations, the eccentricity of the resulting stress diagram (e') is obtained by taking moments about the center of gravity of the original section. The following expression should be used:

$$e' = \frac{\Sigma M + M_u}{\Sigma W - \overline{A'4} \cdot T_1}$$

where:

ΣM = summation of moments of all external forces

M_u = moment of the tentative uplift force ($\overline{A'4} \cdot T_1$)

ΣW = summation of vertical forces

$\overline{A'4}$ = internal hydrostatic pressure ordinate at the end of the crack (see figure 5(c))

T_1 = remaining uncracked portion of the thickness

To compute a revised T_1 based on the e' value obtained from the above expression, use the following expression:

$$T_1 = 3 \left(\frac{T}{2} - e' \right)$$

where:

T = the original thickness (see fig. 5(a)).

After the value of T_1 has been determined with sufficient accuracy (0.5 percent change from the previous value), the resulting stress at the downstream face ($\overline{B5}$ on fig. 5(d)) should be computed using the following expression:

$$\overline{B5} = \frac{2(\Sigma W - (\overline{A'4} \cdot T_1))}{T_1} + \overline{A'4}$$

where all terms are as described above.

The shear-friction factor of safety should be computed for the remaining uncracked portion, T_1 . The remaining uncracked area should be used in determining total cohesion, and $(\Sigma W - \overline{A'4} \cdot T_1)$ should be used as the summation of normal forces. Safety requirements for both stress and sliding stability are discussed under "Factors of Safety" in this monograph.

Criteria.—Cracking should be assumed to occur in a gravity dam for the "Extreme Loading Combinations" whenever the vertical normal stress computed at the face does not meet the criteria established under "Factors of Safety" in this monograph. The crack should be assumed to extend to the point where the compressive stress is equal to the internal hydrostatic pressure within the uncracked portion at the end of the crack. Uplift pressures in the crack when it is opened by earthquake should be assumed to be zero.

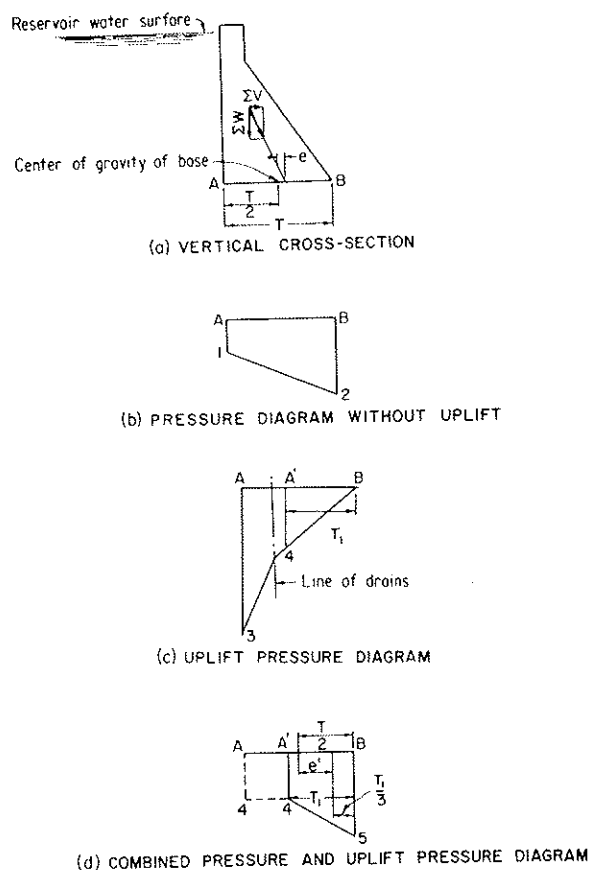


Figure 5.—Diagrams of base pressures acting on a gravity dam.

FACTORS OF SAFETY

Basic Considerations.—All loads to be used in design should be chosen to represent, as nearly as can be determined, the actual loads that will occur on the structure during operation, in accordance with the criteria under "Load Combinations." Methods of determining load-resisting capacity of the dam should be the most accurate available. All uncertainties regarding loads or load-carrying capacity should be resolved as far as practicable by field or laboratory tests and by thorough exploration and inspection of the foundation. Thus, the factor of safety should be as accurate an evaluation as possible of the capacity of the structure to resist applied loads. All safety factors listed are minimum values.

Like other important structures, dams should be regularly and frequently inspected. Adequate observations and measurements should be made of the structural behavior of the dam and its foundation to assure that the structure is functioning as designed.

Although somewhat lower safety factors may be permitted for limited local areas within the foundation, overall safety factors for the dam and its foundation after beneficiation should meet requirements for the loading combination being analyzed.

For other loading combinations where safety factors are not specified, the designer is responsible for selection of safety factors consistent with those for loading combination categories previously discussed. Somewhat higher safety factors should be used for foundation studies because of the greater amount of uncertainty involved in assessing foundation load-resisting capacity.

Safety factors for gravity dams are based on the use of the gravity method of analysis and those for foundation sliding stability are based on an assumption of uniform stress distribution on the plane being analyzed.

Criteria.—(1) *Compressive stress.*—The maximum allowable compressive stress for concrete in a gravity dam subjected to any of the "Usual Loading Combinations" should not be greater than the specified compressive strength divided by a safety factor of 3.0. Under no circumstance should the allowable compressive stress for the "Usual Loading Combinations" exceed 1,500 lbs/in² (10.3 MPa).

A safety factor of 2.0 should be used in determining the allowable compressive stress for the "Unusual Loading Combinations." The

maximum allowable compressive stress for the "Unusual Loading Combinations" should in no case exceed 2,250 lbs/in² (15.5 MPa). The maximum allowable compressive stress for the "Extreme Loading Combinations" should be determined in the same way using a safety factor greater than 1.0.

Safety factors of 4.0, 2.7, and 1.3 should be used in determining allowable compressive stresses in the foundation for "Usual," "Unusual," and "Extreme Loading Combinations," respectively.

(2) *Tensile stress.*—In order not to exceed the allowable tensile stress, the minimum allowable compressive stress computed without internal hydrostatic pressure should be determined from the following expression which takes into account the tensile strength of the concrete at the lift surfaces:

$$\sigma_{z_u} = p \cdot w \cdot h - (f_t/s)$$

where:

σ_{z_u} = minimum allowable stress at the face

p = a reduction factor to account for drains

w = unit weight of water

h = depth below water surface

f_t = tensile strength of concrete at lift surfaces

s = safety factor.

All parameters must be specified using consistent units.

The value of p should be 1.0 if drains are not present or if cracking occurs at the downstream face and 0.4 if drains are used. A safety factor of 3.0 should be used for "Usual" and 2.0 for "Unusual Loading Combinations." The allowable value of σ_{z_u} for "Usual Loading Combinations" should never be less than 0. Cracking should be assumed to occur if the stress at the upstream face is less than σ_{z_u} computed from the above equation with a safety factor of 1.0 for the "Extreme Loading Combinations." The structure should be deemed safe for this loading if, after cracking has been included, stresses in the structure do not exceed the specified strengths and sliding stability is maintained.

(3) *Sliding stability*.—The shear-friction factor of safety provides a measure of the safety against sliding or shearing on any section. The following expression is the ratio of resisting to driving forces and applies to any section in the structure or at its contact with the foundation for the computation of the shear-friction factor of safety, Q :

$$Q = \frac{CA + (\sum N + \sum U) \tan \phi}{\sum V}$$

where:

- C = unit cohesion
- A = area of section considered
- $\sum N$ = summation of normal forces
- $\sum U$ = summation of uplift forces
- $\tan \phi$ = coefficient of internal friction
- $\sum V$ = summation of shear forces

All parameters must be specified using consistent units. Uplift is negative according to the sign convention in reference [2].

The minimum shear-friction factor within the dam or at the concrete-to-rock contact should be 3.0 for "Usual," 2.0 for "Unusual," and greater than 1.0 for the "Extreme Loading Combinations." The factor of safety against sliding on any plane of weakness within the foundation should not be less than 4.0 for "Usual," 2.7 for "Unusual Loading Combinations," and greater than 1.3 for the "Extreme Loading Combinations" [2]. If the computed safety factor is less than required, foundation treatment can be included to increase the safety factor to the required value.

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