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30 June 1995

US Army Corps
of Engineers

ENGINEERING AND DESIGN

Gravity Dam Design

ENGINEER MANUAL

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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

CECW-ED

Manual
No. 1110-2-2200

30 June 1995

Engineering and Design
GRAVITY DAM DESIGN

1. Purpose. The purpose of this manual is to provide technical criteria and guidance for the planning and design of concrete gravity dams for civil works projects.

2. Applicability. This manual applies to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having responsibilities for the design of civil works projects.

3. Discussion. This manual presents analysis and design guidance for concrete gravity dams. Conventional concrete and roller compacted concrete are both addressed. Curved gravity dams designed for arch action and other types of concrete gravity dams are not covered in this manual. For structures consisting of a section of concrete gravity dam within an embankment dam, the concrete section will be designed in accordance with this manual.

FOR THE COMMANDER:

JAMES D. CRAIG
Colonel, Corps of Engineers
Chief of Staff

This engineer manual supersedes EM 1110-2-2200 dated 25 September 1958.

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Table of Contents

Chapter 1
Introduction

1-1. Purpose

The purpose of this manual is to provide technical criteria and guidance for the planning and design of concrete gravity dams for civil works projects. Specific areas covered include design considerations, load conditions, stability requirements, methods of stress analysis, seismic analysis guidance, and miscellaneous structural features. Information is provided on the evaluation of existing structures and methods for improving stability.

1-2. Scope

a. This manual presents analysis and design guidance for concrete gravity dams. Conventional concrete and roller compacted concrete (RCC) are both addressed. Curved gravity dams designed for arch action and other types of concrete gravity dams are not covered in this manual. For structures consisting of a section of concrete gravity dam within an embankment dam, the concrete section will be designed in accordance with this manual.

b. The procedures in this manual cover only dams on rock foundations. Dams on pile foundations should be designed according to Engineer Manual (EM) 1110-2-2906.

c. Except as specifically noted throughout the manual, the guidance for the design of RCC and conventional concrete dams will be the same.

1-3. Applicability

This manual applies to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having responsibilities for the design of civil works projects.

1-4. References

Required and related publications are listed in Appendix A.

1-5. Terminology

Appendix B contains definitions of terms that relate to the design of concrete gravity dams.

This engineer manual supersedes EM 1110-2-2200 dated 25 September 1958.

Chapter 2
General Design Considerations

2-1. Types of Concrete Gravity Dams

Basically, gravity dams are solid concrete structures that maintain their stability against design loads from the geometric shape and the mass and strength of the concrete. Generally, they are constructed on a straight axis, but may be slightly curved or angled to accommodate the specific site conditions. Gravity dams typically consist of a nonoverflow section(s) and an overflow section or spillway. The two general concrete construction methods for concrete gravity dams are conventional placed mass concrete and RCC.

a. Conventional concrete dams.

(1) Conventionally placed mass concrete dams are characterized by construction using materials and techniques employed in the proportioning, mixing, placing, curing, and temperature control of mass concrete (American Concrete Institute (ACI) 207.1 R-87). Typical overflow and nonoverflow sections are shown on Figures 2-1 and 2-2. Construction incorporates methods that have been developed and perfected over many years of designing and building mass concrete dams. The cement hydration process of conventional concrete limits the size and rate of concrete placement and necessitates building in monoliths to meet crack control requirements. Generally using large-size coarse aggregates, mix proportions are selected to produce a low-slump concrete that gives economy, maintains good workability during placement, develops minimum temperature rise during hydration, and produces important properties such as strength, impermeability, and durability. Dam construction with conventional concrete readily facilitates installation of conduits, penstocks, galleries, etc., within the structure.

(2) Construction procedures include batching and mixing, and transportation, placement, vibration, cooling, curing, and preparation of horizontal construction joints between lifts. The large volume of concrete in a gravity dam normally justifies an onsite batch plant, and requires an aggregate source of adequate quality and quantity, located at or within an economical distance of the project. Transportation from the batch plant to the dam is generally performed in buckets ranging in size from 4 to 12 cubic yards carried by truck, rail, cranes, cableways, or a combination of these methods. The maximum bucket size is usually restricted by the capability of effectively spreading and vibrating the concrete pile after it is dumped from the bucket. The concrete is placed in lifts of 5- to 10-foot depths. Each lift consists of successive layers not exceeding 18 to 20 inches. Vibration is generally performed by large one-man, air-driven, spud-type vibrators. Methods of cleaning horizontal construction joints to remove the weak laitance film on the surface during curing include green cutting, wet sand-blasting, and high-pressure air-water jet. Additional details of conventional concrete placements are covered in EM 1110-2-2000.

(3) The heat generated as cement hydrates requires careful temperature control during placement of mass concrete and for several days after placement. Uncontrolled heat generation could result in excessive tensile stresses due to extreme gradients within the mass concrete or due to temperature reductions as the concrete approaches its annual temperature cycle. Control measures involve precooling and postcooling techniques to limit the peak temperatures and control the temperature drop. Reduction in the cement content and cement replacement with pozzolans have reduced the temperature-rise potential. Crack control is achieved by constructing the conventional concrete gravity dam in a series of individually stable monoliths separated by transverse contraction joints. Usually, monoliths are approximately 50 feet wide. Further details on temperature control methods are provided in Chapter 6.

b. Roller-compacted concrete (RCC) gravity dams. The design of RCC gravity dams is similar to conventional concrete structures. The differences lie in the construction methods, concrete mix design, and details of the appurtenant structures. Construction of an RCC dam is a relatively new and economical concept. Economic advantages are achieved with rapid placement using construction techniques that are similar to those employed for embankment dams. RCC is a relatively dry, lean, zero slump concrete material containing coarse and fine aggregate that is consolidated by external vibration using vibratory rollers, dozer, and other heavy equipment. In the hardened condition, RCC has similar properties to conventional concrete. For effective consolidation, RCC must be dry enough to support the weight of the construction equipment, but have a consistency wet enough to permit adequate distribution of the past binder throughout the mass during the mixing and vibration process and, thus, achieve the necessary compaction of the RCC and prevention of undesirable segregation and voids. The consistency requirements have a direct effect on the mixture proportioning requirements (ACI 207.1 R-87). EM 1110-2-2006, Roller Compacted Concrete, provides detailed

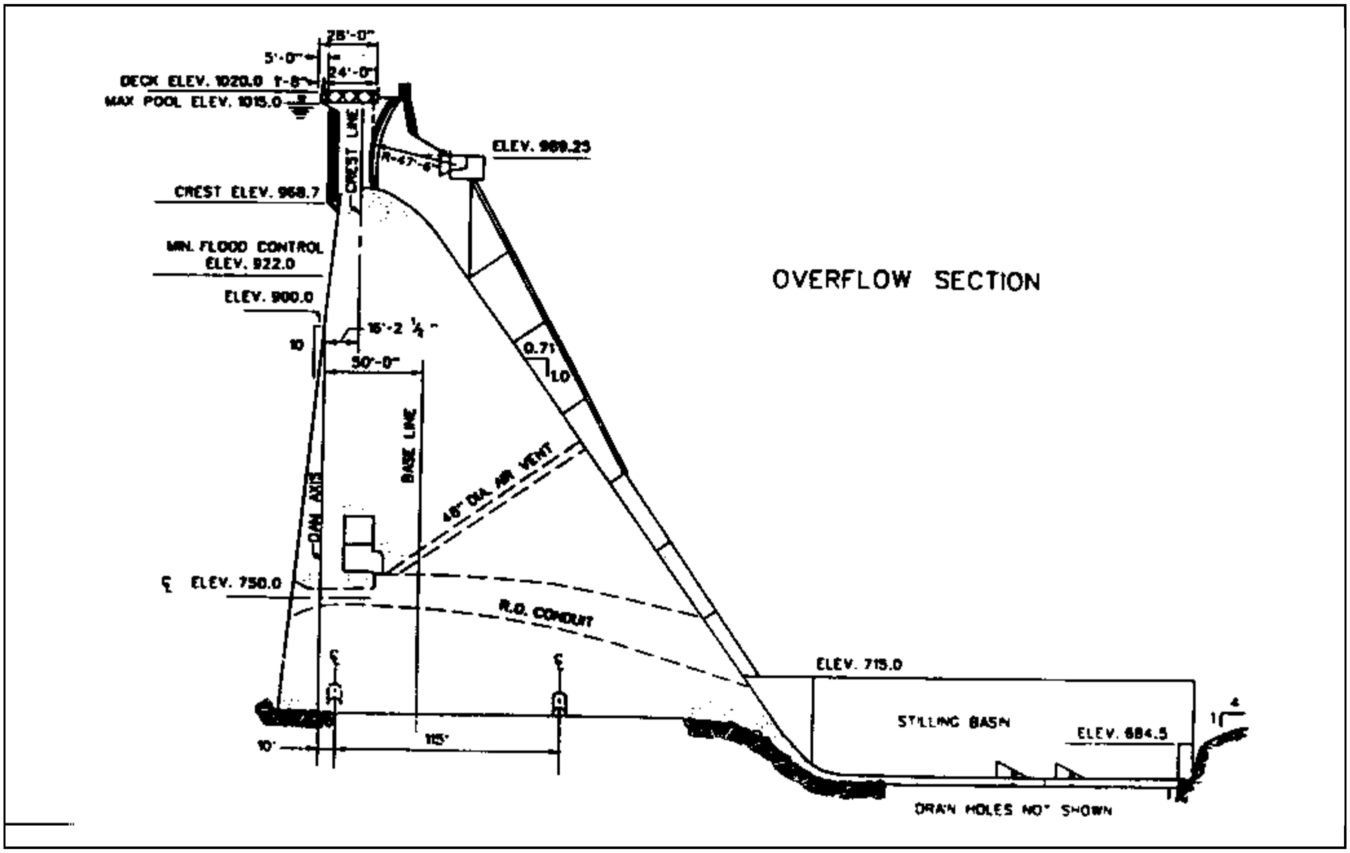

Figure 2-1. Typical dam overflow section

guidance on the use, design, and construction of RCC. Further discussion on the economic benefits and the design and construction considerations is provided in Chapter 9.

2-2. Coordination Between Disciplines

A fully coordinated team of structural, material, and geotechnical engineers, geologists, and hydrological and hydraulic engineers should ensure that all engineering and geological considerations are properly integrated into the overall design. Some of the critical aspects of the analysis and design process that require coordination are:

a. Preliminary assessments of geological data, subsurface conditions, and rock structure. Preliminary designs are based on limited site data. Planning and evaluating field explorations to make refinements in design based on site conditions should be a joint effort of structural and geotechnical engineers.

b. Selection of material properties, design parameters, loading conditions, loading effects, potential failure mechanisms, and other related features of the analytical models. The structural engineer should be involved in these activities to obtain a full understanding of the limits of uncertainty in the selection of loads, strength parameters, and potential planes of failure within the foundation.

c. Evaluation of the technical and economic feasibility of alternative type structures. Optimum structure type and foundation conditions are interrelated. Decisions on alternative structure types to be used for comparative studies need to be made jointly with geotechnical engineers to ensure the technical and economic feasibility of the alternatives.

d. Constructibility reviews in accordance with ER 415-1-11. Participation in constructibility reviews is necessary to ensure that design assumptions and methods of construction are compatible. Constructibility reviews should be followed by a memorandum from the Directorate of Engineering to the Resident Engineer concerning special design considerations and scheduling of construction visits by design engineers during crucial stages of construction.

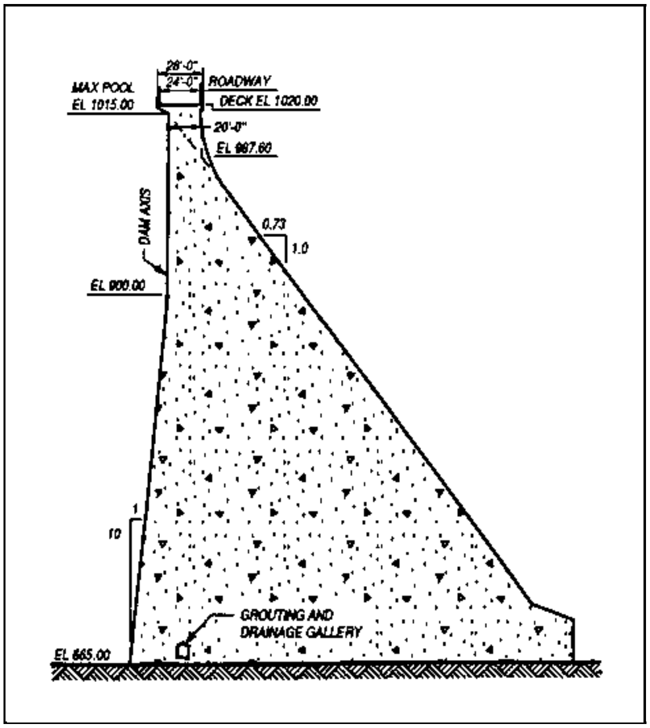

Figure 2-2. Nonoverflow section

e. Refinement of the preliminary structure configuration to reflect the results of detailed site explorations, materials availability studies, laboratory testing, and numerical analysis. Once the characteristics of the foundation and concrete materials are defined, the founding levels of the dam should be set jointly by geotechnical and structural engineers, and concrete studies should be made to arrive at suitable mixes, lift thicknesses, and required crack control measures.

f. Cofferdam and diversion layout, design, and sequencing requirements. Planning and design of these features will be based on economic risk and require the joint effort of hydrologists and geotechnical, construction, hydraulics, and structural engineers. Cofferdams must be set at elevations which will allow construction to proceed with a minimum of interruptions, yet be designed to allow controlled flooding during unusual events.

g. Size and type of outlet works and spillway. The size and type of outlet works and spillway should be set jointly with all disciplines involved during the early stages of design. These features will significantly impact on the configuration of the dam and the sequencing of construction operations. Special hydraulic features such as water quality control structures need to be developed jointly with hydrologists and mechanical and hydraulics engineers.

h. Modification to the structure configuration during construction due to unexpected variations in the foundation conditions. Modifications during construction are costly and should be avoided if possible by a comprehensive exploration program during the design phase. However, any changes in foundation strength or rock structure from those upon which the design is based must be fully evaluated by the structural engineer.

2-3. Construction Materials

The design of concrete dams involves consideration of various construction materials during the investigations phase. An assessment is required on the availability and suitability of the materials needed to manufacture concrete qualities meeting the structural and durability requirements, and of adequate quantities for the volume of concrete in the dam and appurtenant structures. Construction materials include fine and coarse aggregates, cementitious materials, water for washing aggregates, mixing, curing of concrete, and chemical admixtures. One of the most important factors in determining the quality and economy of the concrete is the selection of suitable sources of aggregate. In the construction of concrete dams, it is important that the source have the capability of producing adequate quantitives for the economical production of mass concrete. The use of large aggregates in concrete reduces the cement content. The procedures for the investigation of aggregates shall follow the requirements in EM 1110-2-2000 for mass concrete and EM 1110-2-2006 for RCC.

2-4. Site Selection

a. General. During the feasibility studies, the preliminary site selection will be dependent on the project purposes within the Corps' jurisdiction. Purposes applicable to dam construction include navigation, flood damage reduction, hydroelectric power generation, fish and wildlife enhancement, water quality, water supply, and recreation. The feasibility study will establish the most suitable and economical location and type of structure. Investigations will be performed on hydrology and meteorology, relocations, foundation and site geology, construction materials, appurtenant features, environmental considerations, and diversion methods.

b. Selection factors.

(1) A concrete dam requires a sound bedrock foundation. It is important that the bedrock have adequate shear strength and bearing capacity to meet the necessary stability requirements. When the dam crosses a major fault or shear zone, special design features (joints, monolith lengths, concrete zones, etc.) should be incorporated in the design to accommodate the anticipated movement. All special features should be designed based on analytical techniques and testing simulating the fault movement. The foundation permeability and the extent and cost of foundation grouting, drainage, or other seepage and uplift control measures should be investigated. The reservoir's suitability from the aspect of possible landslides needs to be thoroughly evaluated to assure that pool fluctuations and earthquakes would not result in any mass sliding into the pool after the project is constructed.

(2) The topography is an important factor in the selection and location of a concrete dam and its appurtenant structures. Construction as a site with a narrow canyon profile on sound bedrock close to the surface is preferable, as this location would minimize the concrete material requirements and the associated costs.

(3) The criteria set forth for the spillway, powerhouse, and the other project appurtenances will play an important role in site selection. The relationship and adaptability of these features to the project alignment will need evaluation along with associated costs.

(4) Additional factors of lesser importance that need to be included for consideration are the relocation of existing facilities and utilities that lie within the reservoir and in the path of the dam. Included in these are railroads, powerlines, highways, towns, etc. Extensive and costly relocations should be avoided.

(6) The method or scheme of diverting flows around or through the damsite during construction is an important consideration to the economy of the dam. A concrete gravity dam offers major advantages and potential cost savings by providing the option of diversion through alternate construction blocks, and lowers risk and delay if overtopping should occur.

2-5. Determining Foundation Strength Parameters

a. General. Foundation strength parameters are required for stability analysis of the gravity dam section. Determination of the required parameters is made by evaluation of the most appropriate laboratory and/or in situ strength tests on representative foundation samples coupled with extensive knowledge of the subsurface geologic characteristics of a rock foundation. In situ testing is expensive and usually justified only on very large projects or when foundation problems are know to exist. In situ testing would be appropriate where more precise foundation parameters are required because rock strength is marginal or where weak layers exist and in situ properties cannot be adequately determined from laboratory testing of rock samples.

b. Field investigation. The field investigation must be a continual process starting with the preliminary geologic review of known conditions, progressing to a detailed drilling program and sample testing program, and concluding at the end of construction with a safe and operational structure. The scope of investigation and sampling should be based on an assessment of homogeneity or complexity of geological structure. For example, the extent of the investigation could vary from quite limited (where the foundation material is strong even along the weakest potential failure planes) to quite extensive and detailed (where weak zones or seams exist). There is a certain minimum level of investigation necessary to determine that weak zones are not present in the foundation. Field investigations must also evaluate depth and severity of weathering, ground-water conditions (hydrogeology), permeability, strength, deformation characteristics, and excavatibility. Undisturbed samples are required to determine the engineering properties of the foundation materials, demanding extreme care in application and sampling methods. Proper sampling is a combination of science and art; many procedures have been standardized, but alteration and adaptation of techniques are often dictated by specific field procedures as discussed in EM 1110-2-1804.

c. Strength testing. The wide variety of foundation rock properties and rock structural conditions preclude a standardized universal approach to strength testing. Decisions must be made concerning the need for in situ testing. Before any rock testing is initiated, the geotechnical engineer, geologist, and designer responsible for formulating the testing program must clearly define what the purpose of each test is and who will supervise the testing. It is imperative to use all available data, such as results from geological and geophysical studies, when selecting representative samples for testing. Laboratory testing must attempt to duplicate the actual anticipated loading situations as closely as possible. Compressive strength testing and direct shear testing are normally required to determine design values for shear strength and bearing capacity. Tensile strength testing in some cases as well as consolidation and slakeability testing may also be necessary for soft rock foundations. Rock testing procedures are discussed in the *Rock Testing Handbook* (US Army Engineer Waterways Experiment Station (WES) 1980) and in the International Society of Rock Mechanics, "Suggested Methods for Determining Shear Strength," (International Society of Rock Mechanics 1974). These testing methods may be modified as appropriate to fit the circumstances of the project.

d. Design shear strengths. Shear strength values used in sliding analyses are determined from available laboratory and field tests and judgment. For preliminary designs, appropriate shear strengths for various types of rock may be obtained from numerous available references including the US Bureau of Reclamation Reports SP-39 and REC-ERC-74-10, and many reference texts (see bibliography). It is important to select the types of strengthtests to be performed based upon the probable mode of failure. Generally, strengths on rock discontinuities would be used for the active wedge and beneath the structure. A combination of strengths on discontinuities and/or intact rock strengths would be used for the passive wedge when included in the analysis. Strengths along preexisting shear planes (or faults) should be determined from residual shear tests, whereas the strength along other types of discontinuities must consider the strain characteristics of the various materials along the failure plane as well as the effect of asperities.

Chapter 3
Design Data

3-1. Concrete Properties

a. General. The specific concrete properties used in the design of concrete gravity dams include the unit weight, compressive, tensile, and shear strengths, modulus of elasticity, creep, Poisson's ratio, coefficient of thermal expansion, thermal conductivity, specific heat, and diffusivity. These same properties are also important in the design of RCC dams. Investigations have generally indicated RCC will exhibit properties equivalent to those of conventional concrete. Values of the above properties that are to be used by the designer in the reconnaissance and feasibility design phases of the project are available in ACI 207.1R-87 or other existing sources of information on similar materials. Follow-on laboratory testing and field investigations should provide the values necessary in the final design. Temperature control and mix design are covered in EM 1110-2-2000 and Em 1110-2-2006.

b. Strength.

(1) Concrete strength varies with age; the type of cement, aggregates, and other ingredients used; and their proportions in the mixture. The main factor affecting concrete strength is the water-cement ratio. Lowering the ratio improves the strength and overall quality. Requirements for workability during placement, durability, minimum temperature rise, and overall economy may govern the concrete mix proportioning. Concrete strengths should satisfy the early load and construction requirements and the stress criteria described in Chapter 4. Design compressive strengths at later ages are useful in taking full advantage of the strength properties of the cementitious materials and lowering the cement content, resulting in lower ultimate internal temperature and lower potential cracking incidence. The age at which ultimate strength is required needs to be carefully reviewed and revised where appropriate.

(2) Compressive strengths are determined from the standard unconfined compression test excluding creep effects (American Society for Testing and Materials (ASTM) C 39, "Test Method for Compressive Strength of Cylindrical Concrete Specimens"; C 172, "Method of Sampling Freshly Mixed Concrete"; ASTM C 31, "Method of Making and Curing Concrete Test Specimens in the Field").

(3) The shear strength along construction joints or at the interface with the rock foundation can be determined by the linear relationship $T = C + \delta \tan \phi$ in which C is the unit cohesive strength, δ is the normal stress, and $\tan \phi$ represents the coefficient of internal friction.

(4) The splitting tension test (ASTM C 496) or the modulus of rupture test (ASTM C 78) can be used to determine the strength of intact concrete. Modulus of rupture tests provide results which are consistent with the assumed linear elastic behavior used in design. Spitting tension test results can be used; however, the designer should be aware that the results represent nonlinear performance of the sample. A more detailed discussion of these tests is presented in the *ACI Journal* (Raphael 1984).

c. Elastic properties.

(1) The graphical stress-strain relationship for concrete subjected to a continuously increasing load is a curved line. For practical purposes, however, the modulus of elasticity is considered a constant for the range of stresses to which mass concrete is usually subjected.

(2) The modulus of elasticity and Poisson's ratio are determined by the ASTM C 469, "Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression."

(3) The deformation response of a concrete dam subjected to sustained stress can be divided into two parts. The first, elastic deformation, is the strain measured immediately after loading and is expressed as the instantaneous modulus of elasticity. The other, a gradual yielding over a long period, is the inelastic deformation or creep in concrete. Approximate values for creep are generally based on reduced values of the instantaneous modulus. When design requires more exact values, creep should be based on the standard test for creep of concrete in compression (ASTM C 512).

d. Thermal properties. Thermal studies are required for gravity dams to assess the effects of stresses induced by temperature changes in the concrete and to determine the temperature controls necessary to avoid undesirable cracking. The thermal properties required in the study include thermal conductivity, thermal diffusivity, specific heat, and the coefficient of thermal expansion.

e. Dynamic properties.

(1) The concrete properties required for input into a linear elastic dynamic analysis are the unit weight, Young's modulus of elasticity, and Poisson's ratio. The

concrete tested should be of sufficient age to represent the ultimate concrete properties as nearly as practicable. One-year-old specimens are preferred. Usually, upper and lower bound values of Young's modulus of elasticity will be required to bracket the possibilities.

(2) The concrete properties needed to evaluate the results of the dynamic analysis are the compressive and tensile strengths. The standard compression test (see paragraph 3-1*b*) is acceptable, even though it does not account for the rate of loading, since compression normally does not control in the dynamic analysis. The splitting tensile test or the modulus of rupture test can be used to determine the tensile strength. The static tensile strength determined by the splitting tensile test may be increased by 1.33 to be comparable to the standard modulus of rupture test.

(3) The value determined by the modulus of rupture test should be used as the tensile strength in the linear finite element analysis to determine crack initiation within the mass concrete. The tensile strength should be increased by 50 percent when used with seismic loading to account for rapid loading. When the tensile stress in existing dams exceeds 150 percent of the modulus of rupture, nonlinear analyses will be required in consultation with CECW-ED to evaluate the extent of cracking. For initial design investigations, the modulus of rupture can be calculated from the following equation (Raphael 1984):

$$f_t = 2.3 f_c'^{\,2/3} \qquad (3\text{-}1)$$

where

f_t = tensile strength, psi (modulus of rupture)

f_c' = compressive strength, psi

3-2. Foundation Properties

a. Deformation modulus. The deformation modulus of a foundation rock mass must be determined to evaluate the amount of expected settlement of the structure placed on it. Determination of the deformation modulus requires coordination of geologists and geotechnical and structural engineers. The deformation modulus may be determined by several different methods or approaches, but the effect of rock inhomogeneity (due partially to rock discontinuities) on foundation behavior must be accounted for. Thus, the determination of foundation compressibility should consider both elastic and inelastic (plastic) deformations. The resulting "modulus of deformation" is a lower value than the elastic modulus of intact rock. Methods for evaluating foundation moduli include in situ (static) testing (plate load tests, dilatometers, etc.); laboratory testing (uniaxial compression tests, ASTM C 3148; and pulse velocity test, ASTM C 2848); seismic field testing; empirical data (rock mass rating system, correlations with unconfined compressive strength, and tables of typical values); and back calculations using compression measurements from instruments such as a borehole extensometer. The foundation deformation modulus is best estimated or evaluated by in situ testing to more accurately account for the natural rock discontinuities. Laboratory testing on intact specimens will yield only an "upper bound" modulus value. If the foundation contains more than one rock type, different modulus values may need to be used and the foundation evaluated as a composite of two or more layers.

b. Static strength properties. The most important foundation strength properties needed for design of concrete gravity structures are compressive strength and shear strength. Allowable bearing capacity for a structure is often selected as a fraction of the average foundation rock compressive strength to account for inherent planes of weakness along natural joints and fractures. Most rock types have adequate bearing capacity for large concrete structures unless they are soft sedimentary rock types such as mudstones, clayshale, etc.; are deeply weathered; contain large voids; or have wide fault zones. Foundation rock shear strength is given as two values: cohesion (c) and internal friction (ϕ). Design values for shear strength are generally selected on the basis of laboratory direct shear test results. Compressive strength and tensile strength tests are often necessary to develop the appropriate failure envelope during laboratory testing. Shear strength along the foundation rock/structure interface must also be evaluated. Direct shear strength laboratory tests on composite grout/rock samples are recommended to assess the foundation rock/structure interface shear strength. It is particularly important to determine strength properties of discontinuities and the weakest foundation materials (i.e., soft zones in shears or faults), as these will generally control foundation behavior.

c. Dynamic strength properties.

(1) When the foundation is included in the seismic analysis, elastic moduli and Poisson's ratios for the foundation materials are required for the analysis. If the foundation mass is modeled, the rock densities are also required.

(2) Determining the elastic moduli for a rock foundation should include several different methods or approaches, as defined in paragraph 3-2*a*.

(3) Poisson's ratios should be determined from uniaxial compression tests, pulse velocity tests, seismic field tests, or empirical data. Poisson's ratio does not vary widely for rock materials.

(4) The rate of loading effect on the foundation modulus is considered to be insignificant relative to the other uncertainties involved in determining rock foundation properties, and it is not measured.

(5) To account for the uncertainties, a lower and upper bound for the foundation modulus should be used for each rock type modeled in the structural analysis.

3-3. Loads

a. General. In the design of concrete gravity dams, it is essential to determine the loads required in the stability and stress analysis. The following forces may affect the design:

(1) Dead load.

(2) Headwater and tailwater pressures.

(3) Uplift.

(4) Temperature.

(5) Earth and silt pressures.

(6) Ice pressure.

(7) Earthquake forces.

(8) Wind pressure.

(9) Subatmospheric pressure.

(10) Wave pressure.

(11) Reaction of foundation.

b. Dead load. The unit weight of concrete generally should be assumed to be 150 pounds per cubic foot until an exact unit weight is determined from the concrete materials investigation. In the computation of the dead load, relatively small voids such as galleries are normally not deducted except in low dams, where such voids could create an appreciable effect upon the stability of the structure. The dead loads considered should include the weight of concrete, superimposed backfill, and appurtenances such as gates and bridges.

c. Headwater and tailwater.

(1) General. The headwater and tailwater loadings acting on a dam are determined from the hydrology, meteorology, and reservoir regulation studies. The frequency of the different pool levels will need to be determined to assess which will be used in the various load conditions analyzed in the design.

(2) Headwater.

(a) The hydrostatic pressure against the dam is a function of the water depth times the unit weight of water. The unit weight should be taken at 62.5 pounds per cubic foot, even though the weight varies slightly with temperature.

(b) In some cases the jet of water on an overflow section will exert pressure on the structure. Normally such forces should be neglected in the stability analysis except as noted in paragraph 3-3*i*.

(3) Tailwater.

(a) For design of nonoverflow sections. The hydrostatic pressure on the downstream face of a nonoverflow section due to tailwater shall be determined using the full tailwater depth.

(b) For design of overflow sections. Tailwater pressure must be adjusted for retrogression when the flow conditions result in a significant hydraulic jump in the downstream channel, i.e. spillway flow plunging deep into tailwater. The forces acting on the downstream face of overflow sections due to tailwater may fluctuate significantly as energy is dissipated in the stilling basin. Therefore, these forces must be conservatively estimated when used as a stabilizing force in a stability analysis. Studies have shown that the influence of tailwater retrogression can reduce the effective tailwater depth used to calculate pressures and forces to as little as 60 percent of the full tailwater depth. The amount of reduction in the effective depth used to determine tailwater forces is a function of the degree of submergence of the crest of the structure and the backwater conditions in the downstream channel. For new designs, Chapter 7 of EM 1110-2-1603 provides guidance in the calculation of hydraulic pressure distributions in spillway flip buckets due to tailwater conditions.

(c) Tailwater submergence. When tailwater conditions significantly reduce or eliminate the hydraulic jump in the spillway basin, tailwater retrogression can be neglected and 100 percent of the tailwater depth can be used to determine tailwater forces.

(d) Uplift due to tailwater. Full tailwater depth will be used to calculate uplift pressures at the toe of the structure in all cases, regardless of the overflow conditions.

d. Uplift. Uplift pressure resulting from headwater and tailwater exists through cross sections within the dam, at the interface between the dam and the foundation, and within the foundation below the base. This pressure is present within the cracks, pores, joints, and seams in the concrete and foundation material. Uplift pressure is an active force that must be included in the stability and stress analysis to ensure structural adequacy. These pressures vary with time and are related to boundary conditions and the permeability of the material. Uplift pressures are assumed to be unchanged by earthquake loads.

(1) Along the base.

(a) General. The uplift pressure will be considered as acting over 100 percent of the base. A hydraulic gradient between the upper and lower pool is developed between the heel and toe of the dam. The pressure distribution along the base and in the foundation is dependent on the effectiveness of drains and grout curtain, where applicable, and geologic features such as rock permeability, seams, jointing, and faulting. The uplift pressure at any point under the structure will be tailwater pressure plus the pressure measured as an ordinate from tailwater to the hydraulic gradient between upper and lower pool.

(b) Without drains. Where there have not been any provisions provided for uplift reduction, the hydraulic gradient will be assumed to vary, as a straight line, from headwater at the heel to zero or tailwater at the toe. Determination of uplift, at any point on or below the foundation, is demonstrated in Figure 3-1.

(c) With drains. Uplift pressures at the base or below the foundation can be reduced by installing foundation drains. The effectiveness of the drainage system will depend on depth, size, and spacing of the drains; the

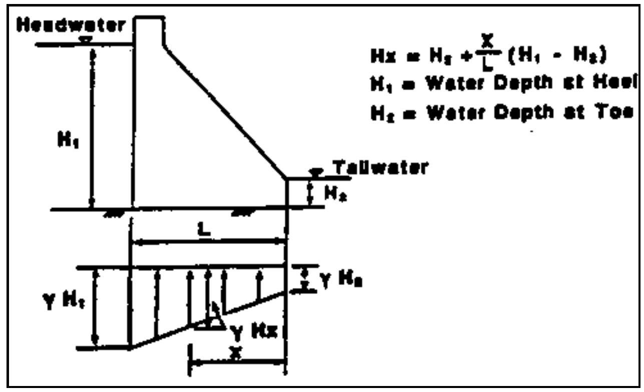

Figure 3-1. Uplift distribution without foundation drainage

character of the foundation; and the facility with which the drains can be maintained. This effectiveness will be assumed to vary from 25 to 50 percent, and the design memoranda should contain supporting data for the assumption used. If foundation testing and flow analysis provide supporting justification, the drain effectiveness can be increased to a maximum of 67 percent with approval from CECW-ED. This criterion deviation will depend on the pool level operation plan instrumentation to verify and evaluate uplift assumptions and an adequate drain maintenance program. Along the base, the uplift pressure will vary linearly from the undrained pressure head at the heel, to the reduced pressure head at the line of drains, to the undrained pressure head at the toe, as shown in Figure 3-2. Where the line of drains intersects the foundation within a distance of 5 percent of the reservoir depth from the upstream face, the uplift may be assumed to vary as a single straight line, which would be the case if the drains were exactly at the heel. This condition is illustrated in Figure 3-3. If the drainage gallery is above tailwater elevation, the pressure of the line of drains should be determined as though the tailwater level is equal to the gallery elevation.

(d) Grout curtain. For drainage to be controlled economically, retarding of flow to the drains from the upstream head is mandatory. This may be accomplished by a zone of grouting (curtain) or by the natural imperviousness of the foundation. A grouted zone (curtain) should be used wherever the foundation is amenable to grouting. Grout holes shall be oriented to intercept the maximum number of rock fractures to maximize its effectiveness. Under average conditions, the depth of the grout zone should be two-thirds to three-fourths of the headwater-tailwater differential and should be supplemented by foundation drain holes with a depth of at least