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30 June 1995

**US Army Corps
of Engineers**

ENGINEERING AND DESIGN

Gravity Dam Design

ENGINEER MANUAL

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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:
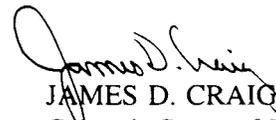
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Table of Contents

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
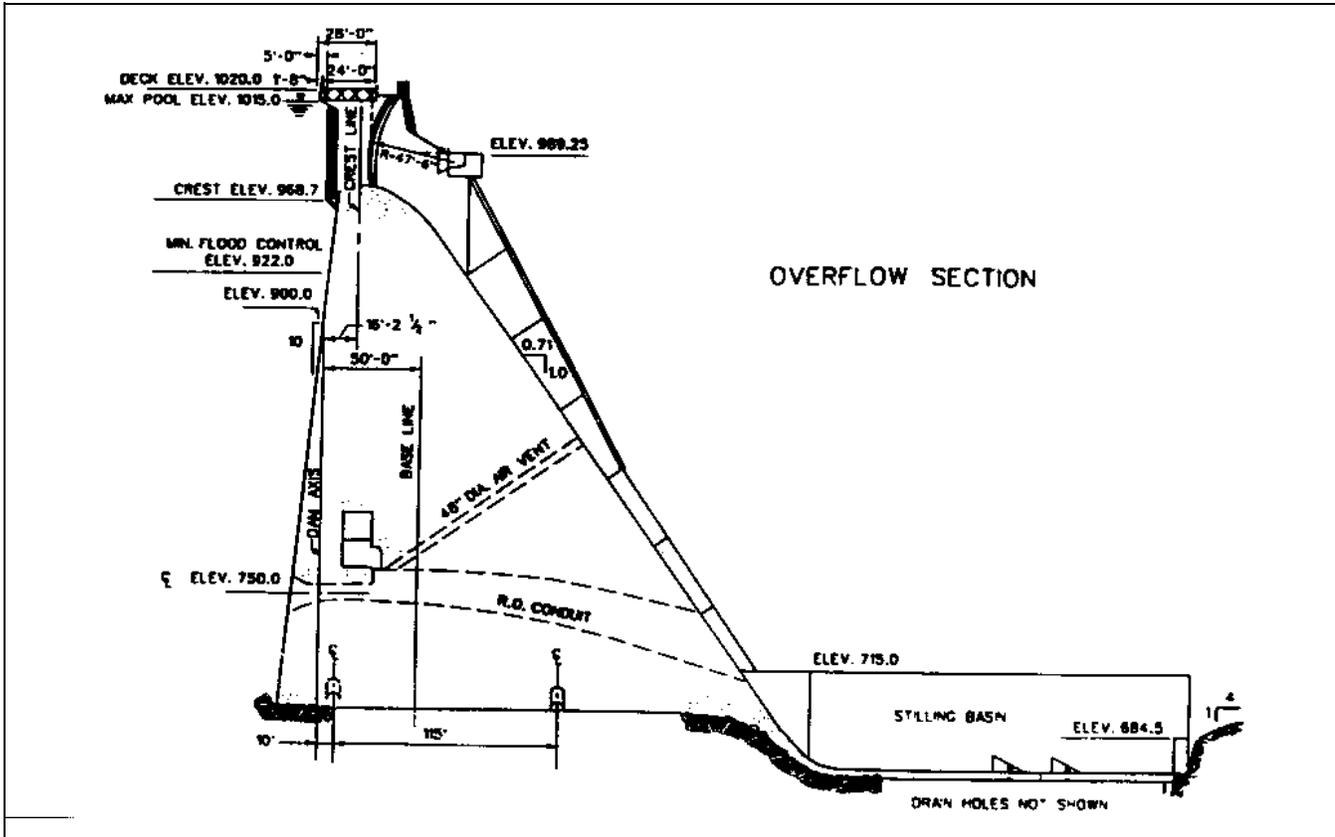
Figure 2-1. Typical dam overflow section

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.
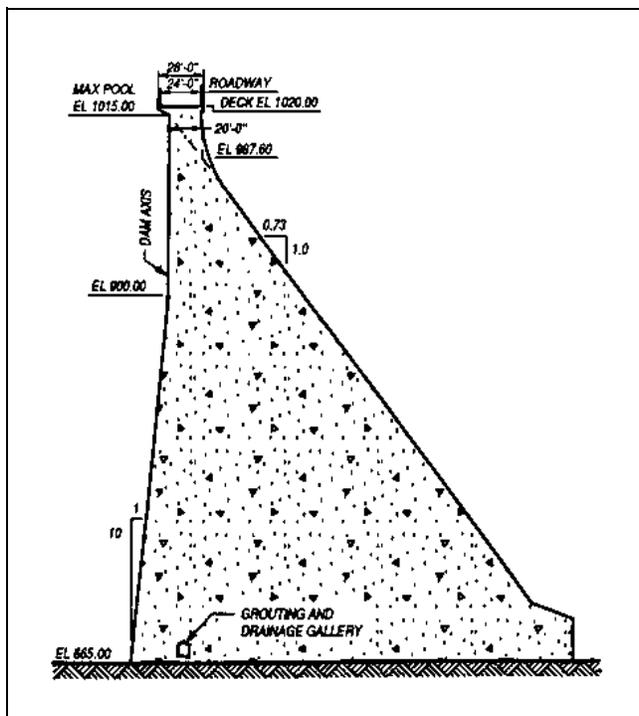
Figure 2-2. Nonoverflow section

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 3-1b) 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
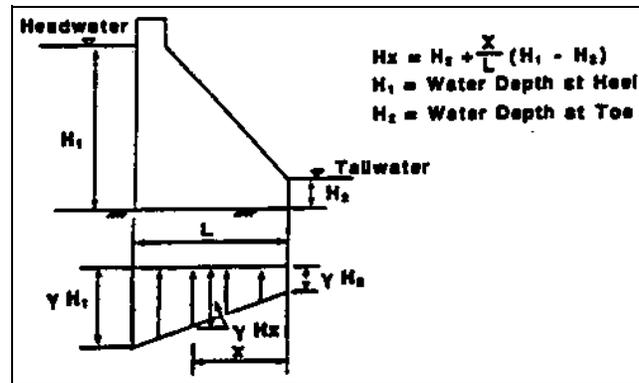

$$H_x = H_2 + \frac{x}{L}(H_1 - H_2)$$
$$H_1 = \text{Water Depth at Heel}$$
$$H_2 = \text{Water Depth at Toe}$$

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
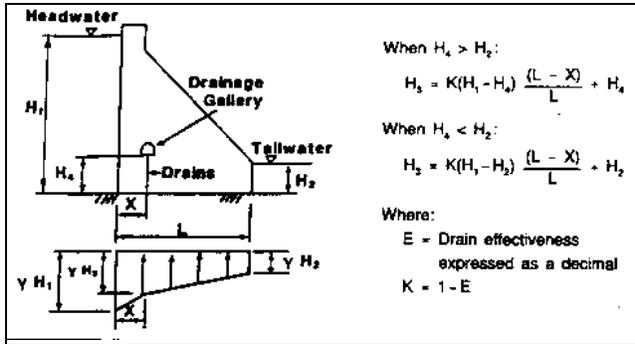
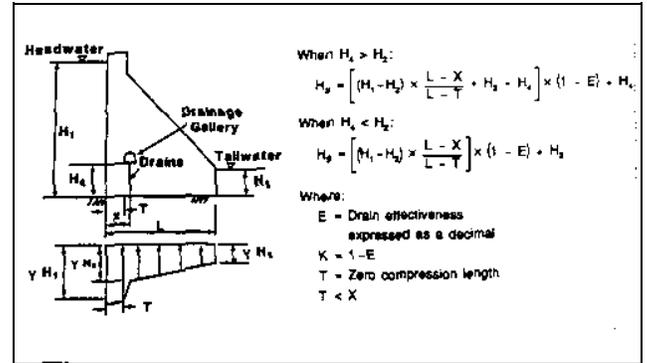**Figure 3-2. Uplift distribution with drainage gallery**

Figure 3-3. Uplift distribution with foundation drains near upstream face

two-thirds that of the grout zone (curtain). Where the foundation is sufficiently impervious to retard the flow and where grouting would be impractical, an artificial cutoff is usually unnecessary. Drains, however, should be provided to relieve the uplift pressures that would build up over a period of time in a relatively impervious medium. In a relatively impervious foundation, drain spacing will be closer than in a relatively permeable foundation.

(e) Zero compression zones. Uplift on any portion of any foundation plane not in compression shall be 100 percent of the hydrostatic head of the adjacent face, except where tension is the result of instantaneous loading resulting from earthquake forces. When the zero compression zone does not extend beyond the location of the drains, the uplift will be as shown in Figure 3-4. For the condition where the zero compression zone extends beyond the drains, drain effectiveness shall not be considered. This uplift condition is shown in Figure 3-5. When an existing dam is being investigated, the design office should submit a request to CECW-ED for a deviation if expensive remedial measures are required to satisfy this loading assumption.

Figure 3-4. Uplift distribution cracked base with drainage, zero compression zone not extending beyond drains

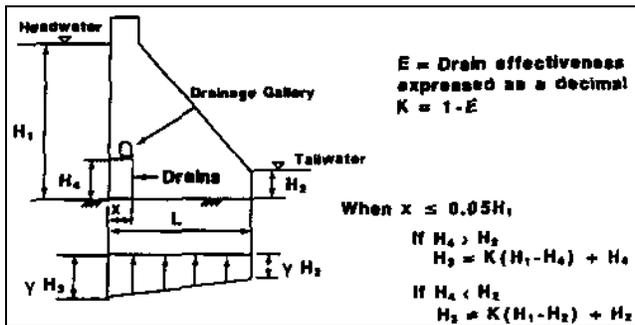**Figure 3-5. Uplift distribution cracked base with drainage, zero compression zone extending beyond drains**

(2) Within dam.

(a) Conventional concrete. Uplift within the body of a conventional concrete-gravity dam shall be assumed to vary linearly from 50 percent of maximum headwater at the upstream face to 50 percent of tailwater, or zero, as the case may be, at the downstream face. This simplification is based on the relative impermeability of intact concrete which precludes the buildup of internal pore pressures. Cracking at the upstream face of an existing dam or weak horizontal construction joints in the body of the dam may affect this assumption. In these cases, uplift along these discontinuities should be determined as described in paragraph 3-3.*d*(1) above.

(b) RCC concrete. The determination of the percent uplift will depend on the mix permeability, lift joint treatment, the placements, techniques specified for minimizing segregation within the mixture, compaction methods, and

the treatment for watertightness at the upstream and downstream faces. A porous upstream face and lift joints in conjunction with an impermeable downstream face may result in a pressure gradient through a cross section of the dam considerably greater than that outlined above for conventional concrete. Construction of a test section during the design phase (in accordance with EM 1110-2-2006, Roller Compacted Concrete) shall be used as a means of determining the permeability and, thereby, the exact uplift force for use by the designer.

(3) In the foundation. Sliding stability must be considered along seams or faults in the foundation. Material in these seams or faults may be gouge or other heavily sheared rock, or highly altered rock with low shear resistance. In some cases, the material in these zones is porous and subject to high uplift pressures upon reservoir filling. Before stability analyses are performed, engineering geologists must provide information regarding potential failure planes within the foundation. This includes the location of zones of low shear resistance, the strength of material within these zones, assumed potential failure planes, and maximum uplift pressures that can develop along the failure planes. Although there are no prescribed uplift pressure diagrams that will cover all foundation failure plane conditions, some of the most common assumptions made are illustrated in Figures 3-6 and 3-7. These diagrams assume a uniform head loss along the failure surface from point "A" to tailwater, and assume that the foundation drains penetrate the failure plane and are effective in reducing uplift on that plane. If there is concern that the drains may be ineffective or partially effective in reducing uplift along the failure plane, then uplift distribution as represented by the dashed line in Figures 3-6 and 3-7 should be considered for stability computations. Dangerous uplift pressures can develop along foundation seams or faults if the material in the seams or faults is pervious and the pervious zone is intercepted by the base of the dam or by an impervious fault. These conditions are described in Casagrande (1961) and illustrated by Figures 3-8 and 3-9. Every effort is made to grout pervious zones within the foundation prior to constructing the dam. In cases where grouting is impractical or ineffective, uplift pressure can be reduced to safe levels through proper drainage of the pervious zone. However, in those circumstances where the drains do not penetrate the pervious zone or where drainage is only partially effective, the uplift conditions shown in Figures 3-8 and 3-9 are possible.
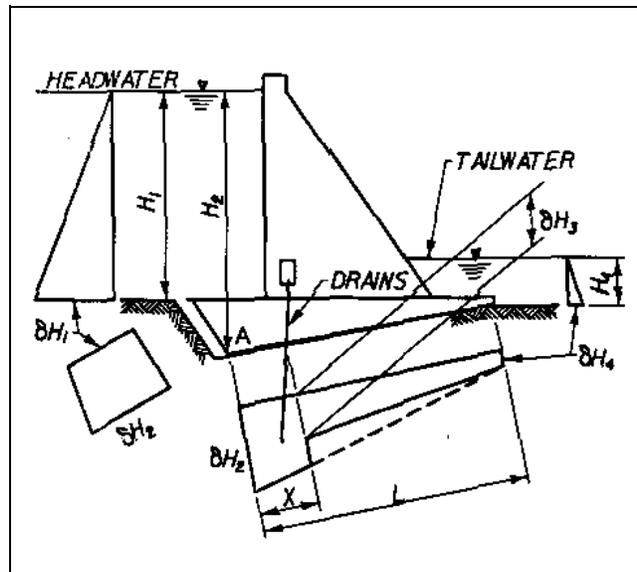

Figure 3-6. Uplift pressure diagram. Dashed line represents uplift distribution to be considered for stability computations
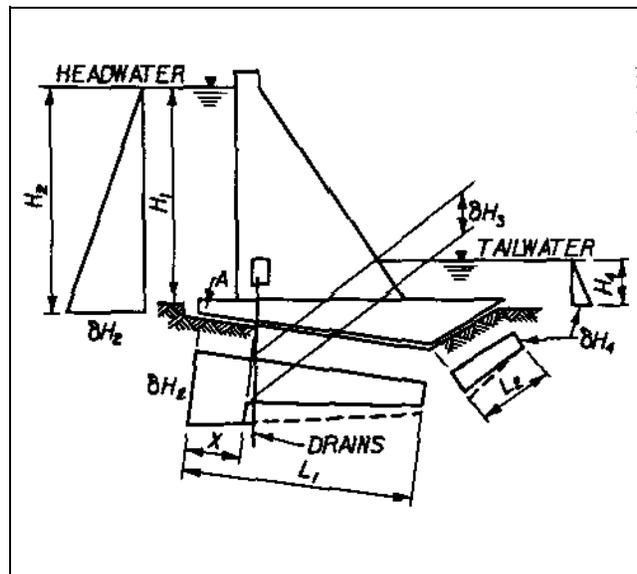

Figure 3-7. Dashed line in uplift pressure diagram represents uplift distribution to be considered for stability computations
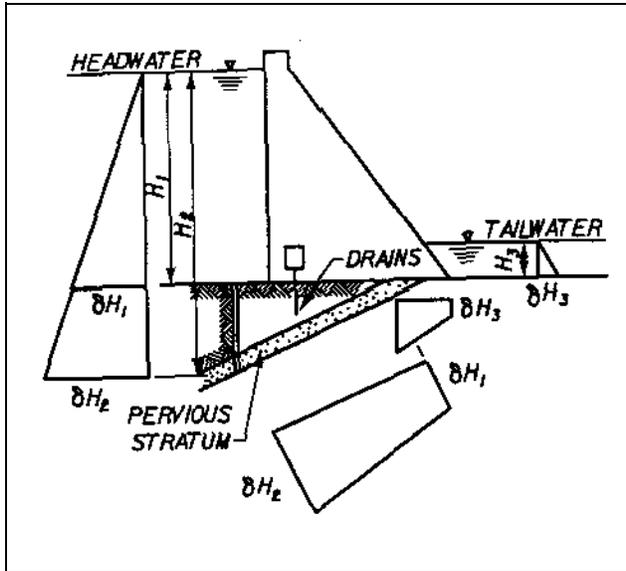
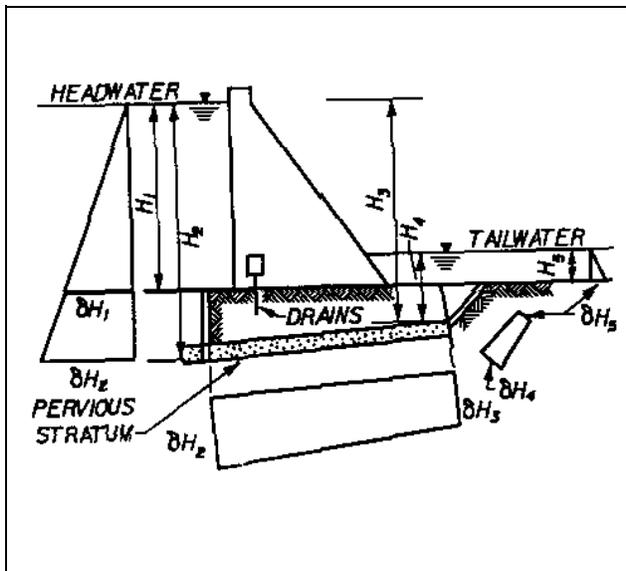**Figure 3-8. Development of dangerous uplift pressure along foundation seams or faults**

Figure 3-9. Effect along foundation seams or faults if material is pervious and pervious zone is intercepted by base of dam or by impervious fault

e. Temperature.

(1) A major concern in concrete dam construction is the control of cracking resulting from temperature change. During the hydration process, the temperature rises because of the hydration of cement. The edges of the monolith release heat faster than the interior; thus the core will be in compression and the edges in tension. When the strength of the concrete is exceeded, cracks will appear on the surface. When the monolith starts cooling, the contraction of the concrete is restrained by the foundation or concrete layers that have already cooled and hardened. Again, if this tensile strain exceeds the capacity of the concrete, cracks will propagate completely through the monolith. The principal concerns with cracking are that it affects the watertightness, durability, appearance, and stresses throughout the structure and may lead to undesirable crack propagation that impairs structural safety.

(2) In conventional concrete dams, various techniques have been developed to reduce the potential for temperature cracking (ACI 224R-80). Besides contraction joints, these include temperature control measures during construction, cements for limiting heat of hydration, and mix designs with increased tensile strain capacity.

(3) If an RCC dam is built without vertical contraction joints, additional internal restraints are present. Thermal loads combined with dead loads and reservoir loads could create tensile strains in the longitudinal axis sufficient to cause transverse cracks within the dam.

f. Earth and silt. Earth pressures against the dam may occur where backfill is deposited in the foundation excavation and where embankment fills abut and wrap around concrete monoliths. The fill material may or may not be submerged. Silt pressures are considered in the design if suspended sediment measurements indicate that such pressures are expected. Whether the lateral earth pressures will be in an active or an at-rest state is determined by the resulting structure lateral deformation. Methods for computing the Earth's pressures are discussed in EM 1110-2-2502, Retaining and Flood Walls.

g. Ice pressure. Ice pressure is of less importance in the design of a gravity dam than in the design of gates and other appurtenances for the dam. Ice damage to the gates is quite common while there is no known instance of any serious ice damage occurring to the dam. For the purpose of design, a unit pressure of not more than 5,000 pounds per square foot should be applied to the contact surface of the structure. For dams in this country, the ice thickness normally will not exceed 2 feet. Climatology studies will determine whether an allowance for ice pressure is appropriate. Further discussion on types of ice/structure interaction and methods for computing ice forces is provided in EM 1110-2-1612, Ice Engineering.

h. Earthquake.

(1) General.

(a) The earthquake loadings used in the design of concrete gravity dams are based on design earthquakes and site-specific motions determined from seismological evaluation. As a minimum, a seismological evaluation should be performed on all projects located in seismic zones 2, 3, and 4. Seismic zone maps of the United States and Territories and guidance for seismic evaluation of new and existing projects during various levels of design documents are provided in ER 1110-2-1806, Earthquake Design and Analysis for Corps of Engineers Projects.

(b) The seismic coefficient method of analysis should be used in determining the resultant location and sliding stability of dams. Guidance for performing the stability analysis is provided in Chapter 4. In strong seismicity areas, a dynamic seismic analysis is required for the internal stress analysis. The criteria and guidance required in the dynamic stress analysis are given in Chapter 5.

(c) Earthquake loadings should be checked for horizontal earthquake acceleration and, if included in the stress analysis, vertical acceleration. While an earthquake acceleration might take place in any direction, the analysis should be performed for the most unfavorable direction.

(2) Seismic coefficient. The seismic coefficient method of analysis is commonly known as the pseudostatic analysis. Earthquake loading is treated as an inertial force applied statically to the structure. The loadings are of two types: inertia force due to the horizontal acceleration of the dam and hydrodynamic forces resulting from the reaction of the reservoir water against the dam (see Figure 3-10). The magnitude of the inertia forces is computed by the principle of mass times the earthquake acceleration. Inertia forces are assumed to act through the center of gravity of the section or element. The seismic coefficient is a ratio of the earthquake acceleration to gravity; it is a dimensionless unit, and in no case can it be related directly to acceleration from a strong motion instrument. The coefficients used are considered to be the same for the foundation and are uniform for the total height of the dam. Seismic coefficients used in design are based on the seismic zones given in ER 1110-2-1806.

(a) Inertia of concrete for horizontal earthquake acceleration. The force required to accelerate the concrete mass of the dam is determined from the equation:
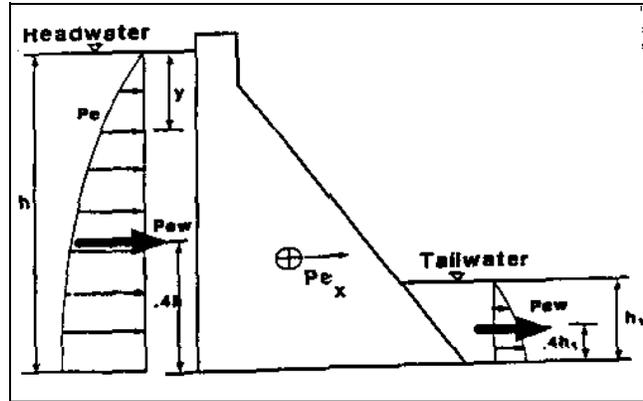
Figure 3-10. Seismically loaded gravity dam, nonoverflow monolith

$$Pe_x = Ma_x = \frac{W}{g} \alpha g = W\alpha \qquad (3\text{-}2)$$

where

Pe_x = horizontal earthquake force

M = mass of dam

a_x = horizontal earthquake acceleration = g

W = weight of dam

g = acceleration of gravity

α = seismic coefficient

(b) Inertia of reservoir for horizontal earthquake acceleration. The inertia of the reservoir water induces an increased or decreased pressure on the dam concurrently with concrete inertia forces. Figure 3-10 shows the pressures and forces due to earthquake by the seismic coefficient method. This force may be computed by means of the Westergaard formula using the parabolic approximation:

$$Pew = \frac{2}{3} Ce \, (\alpha) \, y \, (\sqrt{hy}) \qquad (3\text{-}3)$$

where

Pew = additional total water load down to depth *y* (kips)

Ce = factor depending principally on depth of water and the earthquake vibration period, t_e, in seconds

h = total height of reservoir (feet)

Westergaard's approximate equation for Ce, which is sufficiently accurate for all usual conditions, in pound-second feet units is:

$$Ce = \frac{51}{\sqrt{1 - 0.72 \left(\dfrac{h}{1,000 \, t_e} \right)^2}} \qquad (3\text{-}4)$$

where t_e is the period of vibration.

(3) Dynamic loads. The first step in determining earthquake induced loading involves a geological and seismological investigation of the damsite. The objectives of the investigation are to establish the controlling maximum credible earthquake (MCE) and operating basis earthquake (OBE) and the corresponding ground motions for each, and to assess the possibility of earthquake-produced foundation dislocation at the site. The MCE and OBE are defined in Chapter 5. The ground motions are characterized by the site-dependent design response spectra and, when necessary in the analysis, acceleration-time records. The dynamic method of analysis determines the structural response using either a response spectrum or acceleration-time records for the dynamic input.

(a) Site-specific design response spectra. A response spectrum is a plot of the maximum values of acceleration, velocity, and/or displacement of an infinite series of single-degree-of-freedom systems subjected to an earthquake. The maximum response values are expressed as a function of natural period for a given damping value. The site-specific response spectra is developed statistically from response spectra of strong motion records of earthquakes that have similar source and propagation path properties or from the controlling earthquakes and that were recorded on a similar foundation. Application of the response spectra in dam design is described in Chapter 5.

(b) Acceleration--time records. Accelerograms, used for input for the dynamic analysis, provide a simulation of the actual response of the structure to the given seismic ground motion through time. The acceleration-time records should be compatible with the design response spectrum.

i. Subatmospheric pressure. At the hydrostatic head for which the crest profile is designed, the theoretical pressures along the downstream face of an ogee spillway crest approach atmospheric pressure. For heads higher than the design head, subatmospheric pressures are obtained along the spillway. When spillway profiles are designed for heads appreciably less than the probable maximum that could be obtained, the magnitude of these pressures should be determined and considered in the stability analysis. Methods and discussions covering the determination of these pressures are presented in EM 1110-2-1603, Hydraulic Design of Spillways.

j. Wave pressure. While wave pressures are of more importance in their effect upon gates and appurtenances, they may, in some instances, have an appreciable effect upon the dam proper. The height of waves, runup, and wind setup are usually important factors in determining the required freeboard of any dam. Wave dimensions and forces depend on the extent of water surface or fetch, the wind velocity and duration, and other factors. Information relating to waves and wave pressures are presented in the Coastal Engineering Research Center's *Shore Protection Manual* (SPM), Vol II (SPM 1984).

k. Reaction of foundations. In general, the resultant of all horizontal and vertical forces including uplift must be balanced by an equal and opposite reaction at the foundation consisting of the normal and tangential components. For the dam to be in static equilibrium, the location of this reaction is such that the summation of forces and moments are equal to zero. The distribution of the normal component is assumed as linear, with a knowledge that the elastic and plastic properties of the foundation material and concrete affect the actual distribution.

(1) The problem of determining the actual distribution is complicated by the tangential reaction, internal stress relations, and other theoretical considerations. Moreover, variations of foundation materials with depth, cracks, and fissures that interrupt the tensile and shearing resistance of the foundation also make the problem more complex.

(2) For overflow sections, the base width is generally determined by projecting the spillway slope to the foundation line, and all concrete downstream from this line is disregarded. If a vertical longitudinal joint is not provided at this point, the mass of concrete downstream from the theoretical toe must be investigated for internal stresses.

(3) The unit uplift pressure should be added to the computed unit foundation reaction to determine the maximum unit foundation pressure at any point.

(4) Internal stresses and foundation pressures should be computed with and without uplift to determine the maximum condition.

Chapter 4
Stability Analysis

4-1. Introduction

a. This chapter presents information on the stability analysis of concrete gravity dams. The basic loading conditions investigated in the design and guidance for the dam profile and layout are discussed. The forces acting on a structure are determined as outlined in Chapter 3.

b. For new projects, the design of a gravity dam is performed through an interative process involving a preliminary layout of the structure followed by a stability and stress analysis. If the structure fails to meet criteria then the layout is modified and reanalyzed. This process is repeated until an acceptable cross section is attained. The method for conducting the static and dynamic stress analysis is covered in Chapter 5. The reevaluation of existing structures is addressed in Chapter 8.

c. Analysis of the stability and calculation of the stresses are generally conducted at the dam base and at selected planes within the structure. If weak seams or planes exist in the foundation, they should also be analyzed.

4-2. Basic Loading Conditions

a. The following basic loading conditions are generally used in concrete gravity dam designs (see Figure 4-1). Loadings that are not indicated should be included where applicable. Power intake sections should be investigated with emergency bulkheads closed and all water passages empty under usual loads. Load cases used in the stability analysis of powerhouses and power intake sections are covered in EM 1110-2-3001.

(1) Load Condition No. 1 - unusual loading condition - construction.

(a) Dam structure completed.

(b) No headwater or tailwater.

(2) Load Condition No. 2 - usual loading condition - normal operating.

(a) Pool elevation at top of closed spillway gates where spillway is gated, and at spillway crest where spillway is ungated.

(b) Minimum tailwater.

(c) Uplift.

(d) Ice and silt pressure, if applicable.

(3) Load Condition No. 3 - unusual loading condition - flood discharge.

(a) Pool at standard project flood (SPF).

(b) Gates at appropriate flood-control openings and tailwater at flood elevation.

(c) Tailwater pressure.

(d) Uplift.

(e) Silt, if applicable.

(f) No ice pressure.

(4) Load Condition No. 4 - extreme loading condition - construction with operating basis earthquake (OBE).

(a) Operating basis earthquake (OBE).

(b) Horizontal earthquake acceleration in upstream direction.

(c) No water in reservoir.

(d) No headwater or tailwater.

(5) Load Condition No. 5 - unusual loading condition - normal operating with operating basis earthquake.

(a) Operating basis earthquake (OBE).

(b) Horizontal earthquake acceleration in downstream direction.

(c) Usual pool elevation.
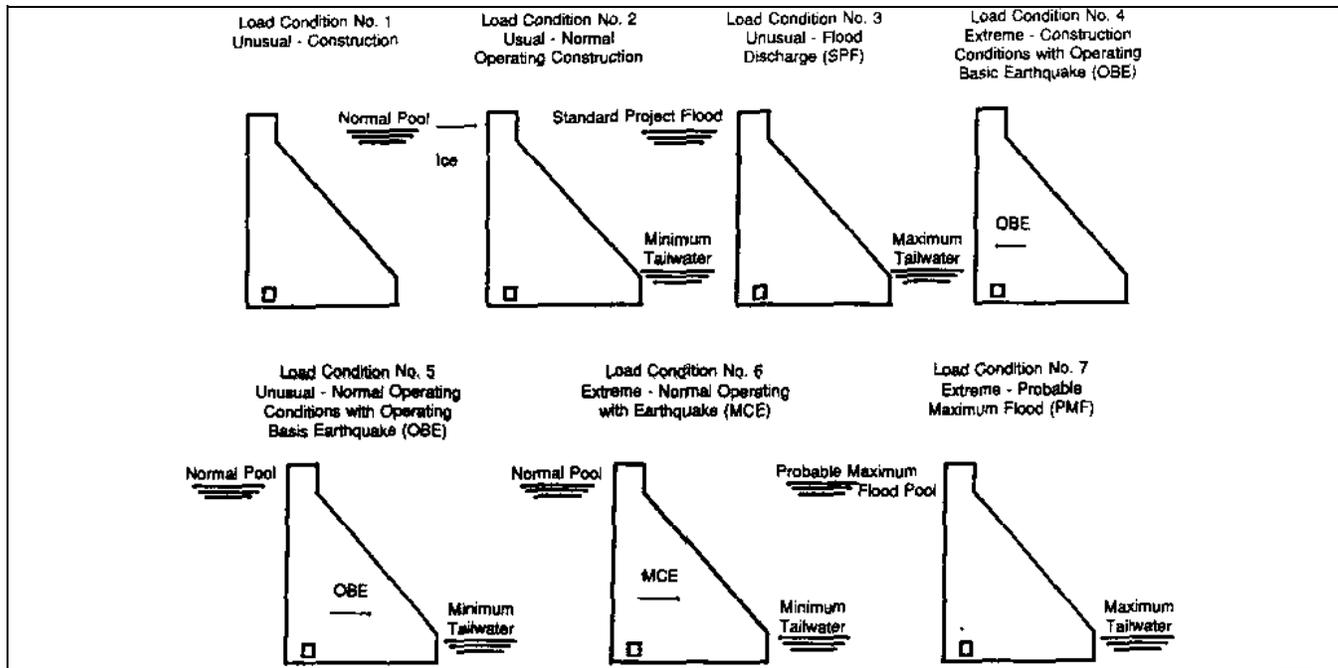
Figure 4-1. Basic loading conditions in concrete gravity dam design

(d) Minimum tailwater.

(e) Uplift at pre-earthquake level.

(f) Silt pressure, if applicable.

(g) No ice pressure.

(6) Load Condition No. 6 - extreme loading condition - normal operating with maximum credible earthquake.

(a) Maximum credible earthquake (MCE).

(b) Horizontal earthquake acceleration in downstream direction.

(c) Usual pool elevation.

(d) Minimum tailwater.

(e) Uplift at pre-earthquake level.

(f) Silt pressure, if applicable.

(g) No ice pressure.

(7) Load Condition No. 7 - extreme loading condition - probable maximum flood.

(a) Pool at probable maximum flood (PMF).

(b) All gates open and tailwater at flood elevation.

(c) Uplift.

(d) Tailwater pressure.

(e) Silt, if applicable.

(f) No ice pressure.

b. In Load Condition Nos. 5 and 6, the selected pool elevation should be the one judged likely to exist coincident with the selected design earthquake event. This means that the pool level occurs, on the average, relatively frequently during the course of the year.

4-3. Dam Profiles

a. Nonoverflow section.

(1) The configuration of the nonoverflow section is usually determined by finding the optimum cross section

that meets the stability and stress criteria for each of the loading conditions. The design cross section is generally established at the maximum height section and then used along the rest of the nonoverflow dam to provide a smooth profile. The upstream face is generally vertical, but may include a batter to increase sliding stability or in existing projects provided to meet prior stability criteria for construction requiring the resultant to fall within the middle third of the base. The downstream face will usually be a uniform slope transitioning to a vertical face near the crest. The slope will usually be in the range of 0.7H to 1V, to 0.8H to 1V, depending on uplift and the seismic zone, to meet the stability requirements.

(2) In the case of RCC dams not using a downstream forming system, it is necessary for construction that the slope not be steeper than 0.8H to 1V and that in applicable locations, it include a sacrificial concrete because of the inability to achieve good compaction at the free edge. The thickness of this sacrificial material will depend on the climatology at the project and the overall durability of the mixture. The weight of this material should not be included in the stability analysis. The upstream face will usually be vertical to facilitate construction of the facing elements. When overstressing of the foundation material becomes critical, constructing a uniform slope at the lower part of the downstream face may be required to reduce foundation pressures. In locations of slope changes, stress concentrations will occur. Stresses should be analyzed in these areas to assure they are within acceptable levels.

(3) The dam crest should have sufficient thickness to resist the impact of floating objects and ice loads and to meet access and roadway requirements. The freeboard at the top of the dam will be determined by wave height and runup. In significant seismicity areas, additional concrete near the crest of the dam results in stress increases. To reduce these stress concentrations, the crest mass should be kept to a minimum and curved transitions provided at slope changes.

b. Overflow section. The overflow or spillway section should be designed in a similar manner as the non-overflow section, complying with stability and stress criteria. The upstream face of the overflow section will have the same configuration as the nonoverflow section. The required downstream face slope is made tangent to the exponential curve of the crest and to the curve at the junction with the stilling basin or flip bucket. The methods used to determine the spillway crest curves is covered in EM 1110-2-1603, Hydraulic Design of Spillways. Piers may be included in the overflow section

to support a bridge crossing the spillway and to support spillway gates. Regulating outlet conduits and gates are generally constructed in the overflow section.

4-4. Stability Considerations

a. General requirements. The basic stability requirements for a gravity dam for all conditions of loading are:

(1) That it be safe against overturning at any horizontal plane within the structure, at the base, or at a plane below the base.

(2) That it be safe against sliding on any horizontal or near-horizontal plane within the structure at the base or on any rock seam in the foundation.

(3) That the allowable unit stresses in the concrete or in the foundation material shall not be exceeded.

Characteristic locations within the dam in which a stability criteria check should be considered include planes where there are dam section changes and high concentrated loads. Large galleries and openings within the structure and upstream and downstream slope transitions are specific areas for consideration.

b. Stability criteria. The stability criteria for concrete gravity dams for each load condition are listed in Table 4-1. The stability analysis should be presented in the design memoranda in a form similar to that shown on Figure 4-1. The seismic coefficient method of analysis, as outlined in Chapter 3, should be used to determine resultant location and sliding stability for the earthquake load conditions. The seismic coefficient used in the analysis should be no less than that given in ER 1110-2-1806, Earthquake Design and Analysis for Corps of Engineers Projects. Stress analyses for a maximum credible earthquake event are covered in Chapter 5. Any deviation from the criteria in Table 4-1 shall be accomplished only with the approval of CECW-ED, and should be justified by comprehensive foundation studies of such nature as to reduce uncertainties to a minimum.

4-5. Overturning Stability

a. Resultant location. The overturning stability is calculated by applying all the vertical forces (ΣV) and lateral forces for each loading condition to the dam and, then, summing moments (ΣM) caused by the consequent forces about the downstream toe. The resultant location along the base is:

Table 4-1
Stability and stress criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress	
				Compressive	Tensile
Usual	Middle 1/3	2.0	≤ allowable	$0.3\ f_c'$	0
Unusual	Middle 1/2	1.7	≤ allowable	$0.5\ f_c'$	$0.6\ f_c'^{2/3}$
Extreme	Within base	1.3	≤ 1.33 × allowable	$0.9\ f_c'$	$1.5\ f_c'^{2/3}$

Note: f_c' is 1-year unconfined compressive strength of concrete. The sliding factors of safety (*FS*) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

$$Resultant\ location = \frac{\Sigma M}{\Sigma V} \qquad (4\text{-}1)$$

$$FS = \frac{\tau_F}{\tau} = \frac{(\sigma \tan \phi + c)}{\tau} \qquad (4\text{-}2)$$

The methods for determining the lateral, vertical, and uplift forces are described in Chapter 3.

b. Criteria. When the resultant of all forces acting above any horizontal plane through a dam intersects that plane outside the middle third, a noncompression zone will result. The relationship between the base area in compression and the location of the resultant is shown in Figure 4-2. For usual loading conditions, it is generally required that the resultant along the plane of study remain within the middle third to maintain compressive stresses in the concrete. For unusual loading conditions, the resultant must remain within the middle half of the base. For the extreme load conditions, the resultant must remain sufficiently within the base to assure that base pressures are within prescribed limits.

4-6. Sliding Stability

a. General. The sliding stability is based on a factor of safety (*FS*) as a measure of determining the resistance of the structure against sliding. The multiple-wedge analysis is used for analyzing sliding along the base and within the foundation. For sliding of any surface within the structure and single planes of the base, the analysis will follow the single plane failure surface of analysis covered in paragraph 4-6*e*.

b. Definition of sliding factor of safety.

(1) The sliding *FS* is conceptually related to failure, the ratio of the shear strength (τ_F), and the applied shear stress (τ) along the failure planes of a test specimen according to Equation 4-2:

where $\tau_F = \sigma \tan \phi + c$, according to the Mohr-Coulomb Failure Criterion (Figure 4-3). The sliding *FS* is applied to the material strength parameters in a manner that places the forces acting on the structure and rock wedges in sliding equilibrium.

(2) The sliding *FS* is defined as the ratio of the maximum resisting shear (T_F) and the applied shear (T) along the slip plane at service conditions:

$$FS = \frac{T_F}{T} = \frac{(N \tan \phi + cL)}{T} \qquad (4\text{-}3)$$

where

N = resultant of forces normal to the assumed sliding plane

ϕ = angle of internal friction

c = cohesion intercept

L = length of base in compression for a unit strip of dam

c. Basic concepts, assumptions, and simplifications.

(1) Limit equilibrium. Sliding stability is based on a limit equilibrium method. By this method, the shear force necessary to develop sliding equilibrium is determined for an assumed failure surface. A sliding mode of failure will occur along the presumed failure surface when the applied shear (T) exceeds the resisting shear (T_F).
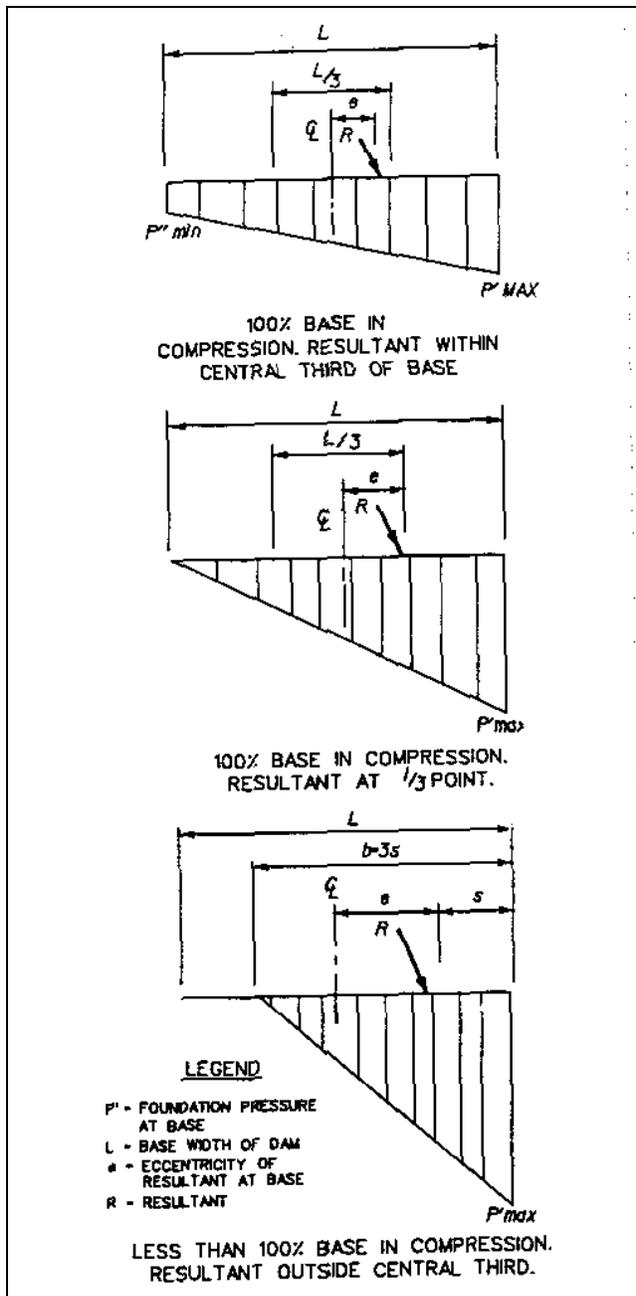
Figure 4-2. Relationship between base area in compression and resultant location

Figure 4-3. Failure envelope

predetermined by discontinuities in the foundation. All the potential planes of failure must be defined and analyzed to determine the one with the least *FS*.

(3) Two-dimensional analysis. The principles presented for sliding stability are based on a two-dimensional analysis. These principles should be extended to a three-dimensional analysis if unique three-dimensional geometric features and loads critically affect the sliding stability of a specific structure.

(4) Force equilibrium only. Only force equilibrium is satisfied in the analysis. Moment equilibrium is not used. The shearing force acting parallel to the interface of any two wedges is assumed to be negligible; therefore, the portion of the failure surface at the bottom of each wedge is loaded only by the forces directly above or below it. There is no interaction of vertical effects between the wedges. The resulting wedge forces are assumed horizontal.

(5) Displacements. Considerations regarding displacements are excluded from the limit equilibrium approach. The relative rigidity of different foundation materials and the concrete structure may influence the results of the sliding stability analysis. Such complex structure-foundation systems may require a more intensive sliding investigation than a limit-equilibrium approach. The effects of strain compatibility along the assumed failure surface may be approximated in the limit-equilibrium approach by selecting the shear strength parameters from in situ or laboratory tests according to the failure strain selected for the stiffest material.

(6) Relationship between shearing and normal forces. A linear relationship is assumed between the resisting shearing force and the normal force acting on the slip plane beneath each wedge. The Coulomb-Mohr Failure Criterion defines this relationship.

(2) Failure surface. The analyses are based on failure surfaces that can be any combination of planes and curves; however, for simplicity all failure surfaces are assumed to be planes. These planes form the bases of the wedges. It should be noted that for the analysis to be realistic, the assumed failure planes have to be kinematically possible. In rock the slip planes may be
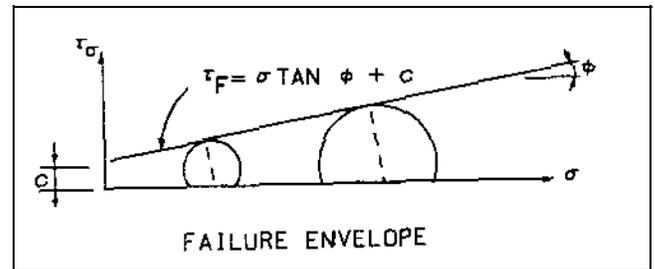
d. Multiple wedge analysis.

(1) General. This method computes the sliding FS required to bring the sliding mass, consisting of the structural wedge and the driving and resisting wedges, into a state of horizontal equilibrium along a given set of slip planes.

(2) Analysis model. In the sliding stability analysis, the gravity dam and the rock and soil acting on the dam are assumed to act as a system of wedges. The dam foundation system is divided into one or more driving wedges, one structural wedge, and one or more resisting wedges, as shown in Figures 4-4 and 4-5.

(3) General wedge equation. By writing equilibrium equations normal and parallel to the slip plane, solving for N_i and T_i, and substituting the expressions for N_i and T_i into the equation for the factor of safety of the typical

wedge, the general wedge and wedge interaction equation can be written as shown in Equation 4-5 (derivation is provided in Appendix C).
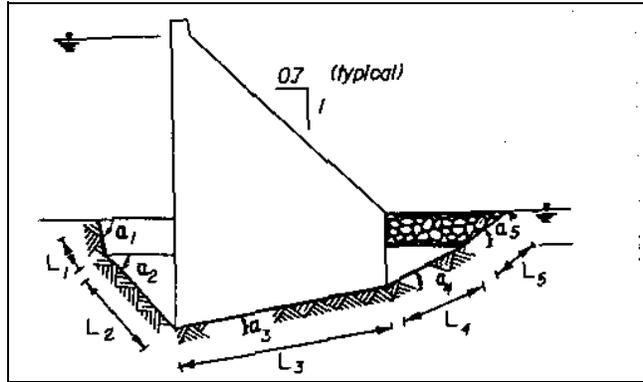
Figure 4-4. Geometry of structure foundation system

$$FS = \left\{ \left[(W_i + V_i) \cos \alpha_i + (H_{Li} - H_{Ri}) \sin \alpha_i + (P_{i-1} - P_i) \sin \alpha_i - U_i \right] \tan \phi_i \right.$$
$$\left. + C_i L_i \right\} / \left[(H_{Li} - H_{Ri}) \cos \alpha_i + (P_{i-1} - P_i) \cos \alpha_i - (W_i + V_i) \sin \alpha_i \right] \tag{4-5}$$
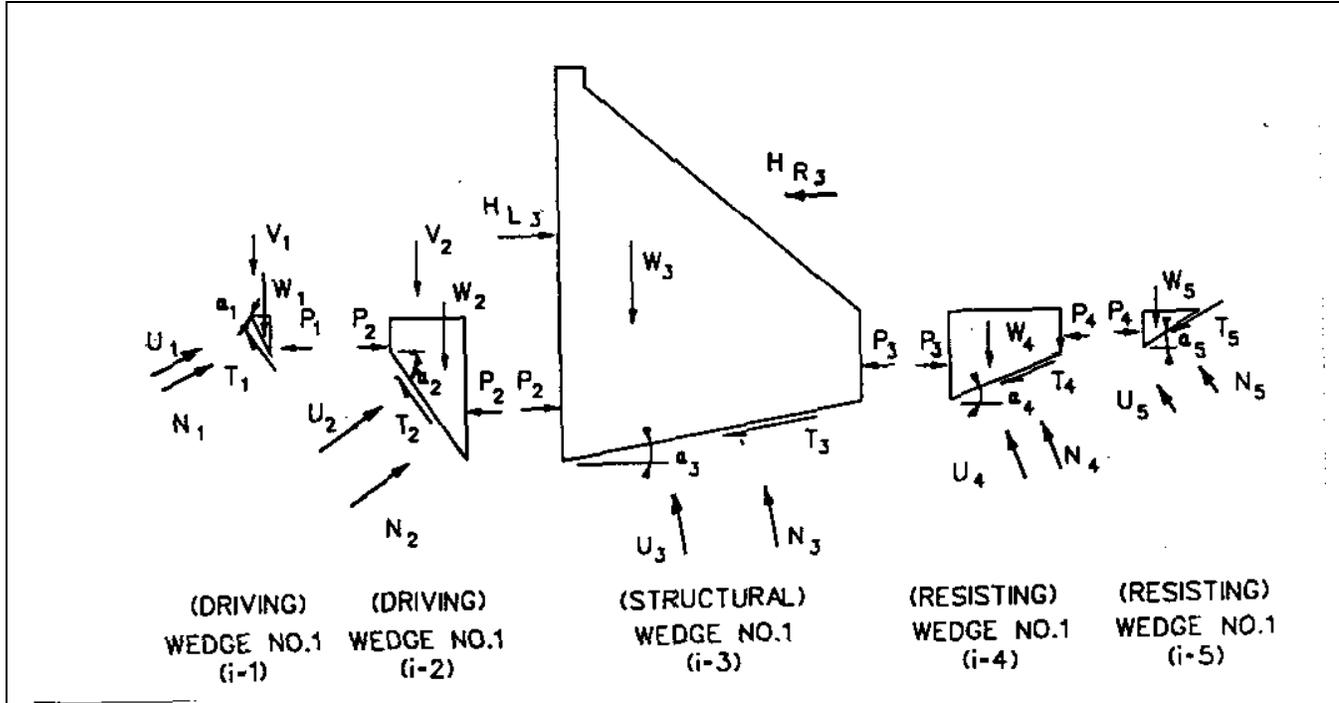

Figure 4-5. Dam foundation system, showing driving, structural, and resisting wedges

Solving for $(P_{i-1} - P_i)$ gives the general wedge equation,

$$(P_{i-1} - P_i) = \left[(W_i + V_i)(\tan \phi_{di} \cos \alpha_i + \sin \alpha_i) - U_i \tan \phi_{di} (H_{Li} - H_{Ri}) \right.$$
$$\left. (\tan \phi_{di} \sin \alpha_i - \cos \alpha_i) + c_{di}L_i \right] \; / \; (\cos \alpha_i - \tan \phi_{di} \sin \alpha_i)$$

(4-6)

where

i = number of wedge being analyzed

$(P_{i-1} - P_i)$ = summation of applied forces acting horizontally on the ith wedge. (A negative value for this term indicates that the applied forces acting on the ith wedge exceed the forces resisting sliding along the base of the wedge. A positive value for the term indicates that the applied forces acting on the ith wedge are less than the forces resisting sliding along the base of that wedge.)

W_i = total weight of water, soil, rock, or concrete in the ith wedge

V_i = any vertical force applied above top of ith wedge

$\tan \phi_{di}$ = $\tan \phi_i$ /FS

α_i = angle between slip plane of ith wedge and horizontal. Positive is counterclockwise

U_i = uplift force exerted along slip plane of the ith wedge

H_{Li} = any horizontal force applied above top or below bottom of left side adjacent wedge

H_{Ri} = any horizontal force applied above top or below bottom of right side adjacent wedge

c_{di} = c_i /FS

L_i = length along the slip plane of the ith wedge

This equation is used to compute the sum of the applied forces acting horizontally on each wedge for an assumed FS. The same FS is used for each wedge. The derivation of the general wedge equation is covered in Appendix C.

(4) Failure plane angle. For the initial trial, the failure plane angle alpha for a driving wedge can be approximated by:

$$\alpha = 45° + \frac{\phi_d}{2}$$

where $\phi_d = \tan^{-1} \left(\frac{\tan \phi}{FS} \right)$

For a resisting wedge, the slip plane angle can be approximated by:

$$\alpha = 45° - \frac{\phi_d}{2}$$

These equations for the slip plane angle are the exact solutions for wedges with a horizontal top surface with or without a uniform surcharge.

(5) Procedure for a multiple-wedge analysis. The general procedure for analyzing multi-wedge systems includes:

(a) Assuming a potential failure surface based on the stratification, location and orientation, frequency and distribution of discontinuities of the foundation material, and the configuration of the base.

(b) Dividing the assumed slide mass into a number of wedges, including a single-structure wedge.

(c) Drawing free body diagrams that show all the forces assuming to be acting on each wedge.

(d) Estimate the FS for the first trial.

(e) Compute the critical sliding angles for each wedge. For a driving wedge, the critical angle is the

angle that produces a maximum driving force. For a resisting wedge, the critical angle is the angle that produces a minimum resisting force.

(f) Compute the uplift pressure, if any, along the slip plane. The effects of seepage and foundation drains should be included.

(g) Compute the weight of each wedge, including any water and surcharges.

(h) Compute the summation of the lateral forces for each wedge using the general wedge equation. In certain cases where the loadings or wedge geometries are complicated, the critical angles of the wedges may not be easily calculated. The general wedge equation may be used to iterate and find the critical angle of a wedge by varying the angle of the wedge to find a minimum resisting or maximum driving force.

(i) Sum the lateral forces for all the wedges.

(j) If the sum of the lateral forces is negative, decrease the *FS* and then recompute the sum of the lateral forces. By decreasing the *FS*, a greater percentage of the shearing strength along the slip planes is mobilized. If the sum of the lateral forces is positive, increase the *FS* and recompute the sum of the lateral forces. By increasing the *FS*, a smaller percentage of the shearing strength is mobilized.

(k) Continue this trial and error process until the sum of the lateral forces is approximately zero for the *FS* used. This procedure will determine the *FS* that causes the sliding mass in horizontal equilibrium, in which the sum of the driving forces acting horizontally equals the sum of the resisting forces that act horizontally.

(l) If the *FS* is less than the minimum criteria, a redesign will be required by sloping or widening the base.

e. Single-plane failure surface. The general wedge equation reduces to Equation 4-7 providing a direct solution for *FS* for sliding of any plane within the dam and for structures defined by a single plane at the interface between the structure and foundation material with no embedment. Figure 4-6 shows a graphical representation of a single-plane failure mode for sloping and horizontal surfaces.
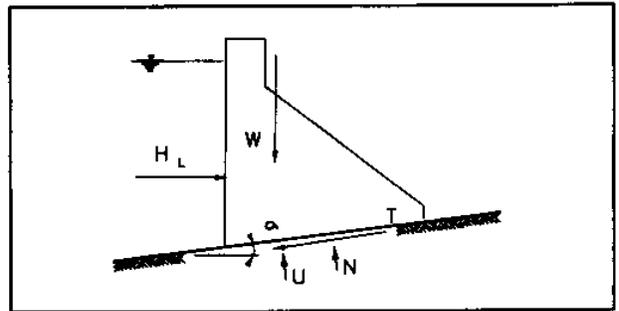
$$FS = \frac{[W \cos \alpha - U + H \sin \alpha] \tan \phi + CL}{H \cos \alpha - W \sin \alpha} \quad (4\text{-}7)$$

where

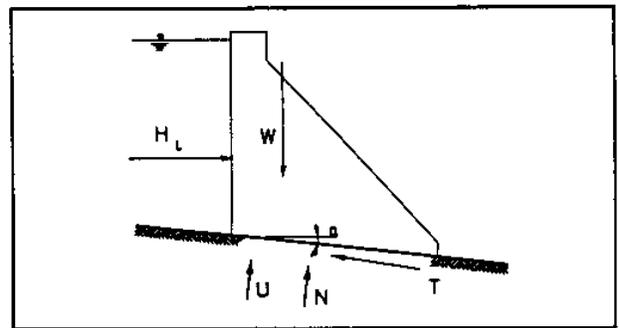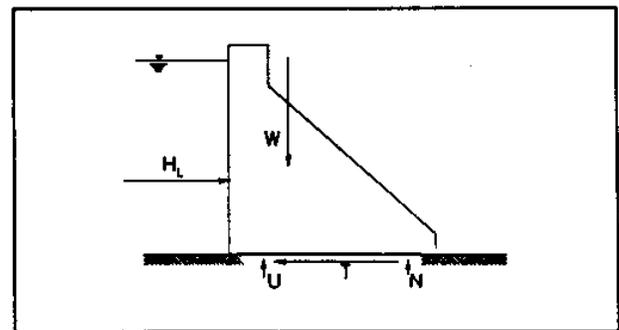H = horizontal force applied to dam

C = cohesion on slip plane

L = length along slip plane

a. Upslope sliding, $\alpha > 0$

b. Downslope slid

c. Horizontal sliding, $\alpha = 0$

Figure 4-6. Single plane failure mode

For the case of sliding through horizontal planes, generally the condition analyzed within the dam, Equation 4-7 reduces to Equation 4-8:

$$FS = \frac{(W - U) \tan \phi + CL}{H_L} \qquad (4\text{-}8)$$

f. Design considerations.

(1) Driving wedges. The interface between the group of driving wedges and the structural wedge is assumed to be a vertical plane that is located at the heel of the structural wedge and extends to its base. The magnitudes of the driving forces depend on the actual values of the safety factor and the inclination angles of the slip path. The inclination angles, corresponding to the maximum active forces for each potential failure surface, can be determined by independently analyzing the group of driving wedges for a trial safety factor. In rock, the inclination may be predetermined by discontinuities in the foundation. The general equation applies directly only to driving wedges with assumed horizontal driving forces.

(2) Structural wedge. The general wedge equation is based on the assumption that shearing forces do not act on the vertical wedge boundaries; hence there can be only one *structural wedge* because concrete structures transmit significant shearing forces across vertical internal planes. Discontinuities in the slip path beneath the structural wedge should be modeled by assuming an average slip plane along the base of the structural wedge.

(3) Resisting wedges. The interface between the group of *resisting wedges* and the structural wedge is assumed to be a vertical plane that is located at the toe of the structural wedge and extends to its base. The magnitudes of the resisting forces depend on the actual values of the safety factor and the inclination angles of the slip path. The inclination angles, corresponding to the minimum passive forces for each potential failure mechanism, can be determined by independently analyzing the group of resisting wedges for a trial safety factor. The general wedge equation applies directly only to resisting wedges with assumed horizontal passive forces. If passive resistance is used, then rock that may be subjected to high velocity water scouring should not be used unless adequately protected. Also, the compressive strength of the rock layers must be sufficient to develop the wedge resistance. In some cases, wedge resistance should not be included unless rock anchors are installed to stabilize the wedge.

(4) Effects of cracks in foundation. Sliding analyses should consider the effects of cracks on the driving side of the structural wedge in the foundation material resulting from differential settlement, shrinkage, or joints in a rock mass. The depth of cracking in massive strong rock foundations should be assumed to extend to the base of the structural wedge. Shearing resistance along the crack should be ignored, and full hydrostatic pressure should be assumed to act at the bottom of the crack. The hydraulic gradient across the base of the structural wedge should reflect the presence of a crack at the heel of the structural wedge.

(5) Uplift. The effects of uplift forces should be included in the sliding analysis. Uplift pressures on the wedges and within any plane within the structure should be determined as described in Chapter 3, Section 3.

(6) Resultant outside kern. As previously stated, requirements for rotational equilibrium are not directly included in the general wedge equation. For some load cases, the normal component of the resultant applied loads will lie outside the kern of the base area, and not all of the structural wedge will be in contact with the foundation material. The sliding analysis should be modified for these load cases to reflect the following secondary effects due to *coupling of the sliding and rational behavior*.

(a) The uplift pressure on the portion of the base not in contact with the foundation material should be a uniform value that is equal to the maximum value of the hydraulic pressure across the base (except for instantaneous load cases such as those resulting from seismic forces).

(b) The cohesive component of the sliding resistance should include only the portion of the base area in contact with the foundation material.

(7) Seismic sliding stability. The sliding stability of a structure for an earthquake-induced base motion should be checked by assuming the specified horizontal earthquake and the vertical earthquake acceleration, if included in the analysis, to act in the most unfavorable direction. The earthquake-induced forces on the structure and foundation wedges may then be determined by the seismic coefficient method as outlined in Chapter 3. Lateral earthquake forces for resisting and driving wedges consisting of soil material should be determined as described in EM 1110-2-2502, Retaining and Flood Walls.

(8) Strain compatibility. Shear resistance in a dam foundation is dependent on the strength properties of the rock. Slide planes within the foundation rock may pass through different materials, and these surfaces may be either through intact rock or along existing rock discontinuities. Less deformation is required for intact rock to reach its maximum shear resistance than for discontinuity surfaces to develop their maximum frictional resistances. Thus, the shear resistance developed along discontinuities depends on the amount of displacement on the intact rock part of the shear surface. If the intact rock breaks, the shear resistance along the entire length of the shear plane is the combined frictional resistance for all materials along the plane.

4-7. Base Pressures

a. Computations of base pressures. For the dam to be in static equilibrium, the resultant of all horizontal and vertical forces including uplift must be balanced by an equal and opposite reaction of the foundation consisting of the total normal reaction and the total tangential shear. The location of this force is such that the summation of moments is equal to zero.

b. Allowable base pressure. The maximum computed base pressure should be equal to or less than the allowable bearing capacity for the usual and unusual load conditions. For extreme loading condition, the maximum bearing pressure should be equal to or less than 1.33 times the allowable bearing capacity.

4-8. Computer Programs

a. Program for sliding stability analysis of concrete structures (CSLIDE).

(1) The computer program CSLIDE has the capability of performing a two-dimensional sliding stability analysis of gravity dams and other concrete structures. It uses the principles of the multi-wedge system of analysis as discussed in paragraph 4-6. Program documentation is covered in U.S. Army Engineer Waterways Experiment Station (WES) Instruction Report ITL-87-5, "Sliding Stability of Concrete Structures (CSLIDE)."

(2) The potential failure planes and the associated wedges are chosen for input and, by satisfying limit equilibrium principles, the *FS* against sliding failure is computed for output. The results also give a summary of failure angles and forces acting on the wedges.

(3) The program considers the effects of:

(a) Multiple layers of rock with irregular surfaces.

(b) Water and seepage effects. The line-of-creep and seepage factor/gradient are provided.

(c) Applied vertical surcharge loads including line, uniform, strip, triangular, and ramp loads.

(d) Applied horizontal concentrated point loads.

(e) Irregularly shaped structural geometry with a horizontal or sloped base.

(f) Percentage of the structure base in compression because of overturning effects.

(g) Single and multiple-plane options for the failure surfaces.

(h) Horizontal and vertical induced loads because of earthquake accelerations.

(i) Factors requiring the user to predetermine the failure surface.

(4) It will not analyze curved surfaces or discontinuities in the slip surface of each wedge. In those cases, an average linear geometry should be assumed along the base of the wedge.

b. Three-dimensional stability analysis and design program (3DSAD), special purpose modules for dams (CDAMS).

(1) General. The computer program called CDAMS performs a three-dimensional stability analysis and design of concrete dams. The program was developed as a specific structure implementation of the three-dimensional stability analysis and design (3DSAD) program. It is intended to handle two cross-sectional types:

(a) An overflow monolith with optional pier.

(b) A nonoverflow monolith.

The program can operate in either an analysis or design mode. Load conditions outlined in paragraph 4-1 can be performed in any order. A more detailed description and information about the use of the program can be found in

Instruction Report K-80-4, "A Three-Dimensional Stability Analysis/Design Program (3DSAD); Report 4, Special Purpose Modules for Dams (CDAMS)" (U.S. Army Corps of Engineers (USACE) 1983).

(2) Analysis. In the analysis mode, the program is capable of performing resultant location, bearing, and sliding computations for each load condition. A review is made of the established criteria and the results outputted.

(3) Design. In the design mode, the structure is incrementally modified until a geometry is established that meets criteria. Different geometric parameters may be varied to achieve a stable geometry. A design memorandum plate option is also available.

Chapter 5
Static and Dynamic Stress Analyses

5-1. Stress Analysis

a. General.

(1) A stress analysis of gravity dams is performed to determine the magnitude and distribution of stresses throughout the structure for static and dynamic load conditions and to investigate the structural adequacy of the substructance and foundation. Load conditions usually investigated are outlined in Chapter 4.

(2) Gravity dam stresses are analyzed by either approximate simplified methods or the finite element method depending on the refinement required for the particular level of design and the type and configuration of the dam. For preliminary designs, simplified methods using cantilever beam models for two-dimensional analysis or the trial load twist method for three-dimensional analysis are appropriate as described in the US Bureau of Reclamation (USBR), "Design of Gravity Dams" (1976). The finite element method is ordinarily used for the feature and final design stages if a more exact stress investigation is required.

b. Finite element analysis.

(1) Finite element models are used for linear elastic static and dynamic analyses and for nonlinear analyses that account for interaction of the dam and foundation. The finite element method provides the capability of modeling complex geometries and wide variations in material properties. The stresses at corners, around openings, and in tension zones can be approximated with a finite element model. It can model concrete thermal behavior and couple thermal stresses with other loads. An important advantage of this method is that complicated foundations involving various materials, weak joints on seams, and fracturing can be readily modeled. Special purpose computer programs designed specifically for analysis of concrete gravity dams are CG-DAMS (Anatech 1993), which performs static, dynamic, and nonlinear analyses and includes a smeared crack model, and MER-LIN (Saouma 1994), which includes a discrete cracking fracture mechanics model.

(2) Two-dimensional, finite element analysis is generally appropriate for concrete gravity dams. The designer should be aware that actual structure response is three-dimensional and should review the analytical and realistic results to assure that the two-dimension approximation is acceptable and realistic. For long conventional concrete dams with transverse contraction joints and without keyed joints, a two-dimensional analysis should be reasonably correct. Structures located in narrow valleys between steep abutments and dams with varying rock moduli which vary across the valley are conditions that necessitate three-dimensional modeling.

(3) The special purpose programs Earthquake Analysis of Gravity Dams including Hydrodynamic Interaction (EADHI) (Chakrabarti and Chopra 1973) and Earthquake Response of Concrete Gravity Dams Including Hydrodynamic and Foundation Interaction Effects (EAGD84) (Chopra, Chakrabarti, and Gupta 1980) are available for modeling the dynamic response of linear two-dimensional structures. Both programs use acceleration time records for dynamic input. The program SDOFDAM is a two-dimensional finite element model (Cole and Cheek 1986) that computes the hydrodynamic loading using Chopra's simplified procedure. The finite element programs such as GTSTRUDL, SAP, ANSYS, ADINA, and ABAQUS provide general capabilities for modeling static and dynamic responses.

5-2. Dynamic Analysis

The structural analysis for earthquake loadings consists of two parts: an approximate resultant location and sliding stability analysis using an appropriate seismic coefficient (see Chapter 4) and a dynamic internal stress analysis using site-dependent earthquake ground motions if the following conditions exist:

a. The dam is 100 feet or more in height and the peak ground acceleration (PGA) at the site is greater than 0.2 g for the maximum credible earthquake.

b. The dam is less than 100 feet high and the PGA at the site is greater than 0.4 g for the maximum credible earthquake.

c. There are gated spillway monoliths, wide roadways, intake structures, or other monoliths of unusual shape or geometry.

d. The dam is in a weakened condition because of accident, aging, or deterioration. The requirements for a dynamic stress analysis in this case will be decided on a project-by-project basis in consultant and approved by CECW-ED.

5-3. Dynamic Analysis Process

The procedure for performing a dynamic analysis include the following:

a. Review the geology, seismology, and contemporary tectonic setting.

b. Determine the earthquake sources.

c. Select the candidate maximum credible and operating basis earthquake magnitudes and locations.

d. Select the attenuation relationships for the candidate earthquakes.

e. Select the controlling maximum credible and operating basis earthquakes from the candidate earthquakes based on the most severe ground motions at the site.

f. Select the design response spectra for the controlling earthquakes.

g. Select the appropriate acceleration-time records that are compatible with the design response spectra if acceleration-time history analyses are needed.

h. Select the dynamic material properties for the concrete and foundation.

i. Select the dynamic methods of analysis to be used.

j. Perform the dynamic analysis.

k. Evaluate the stresses from the dynamic analysis.

5-4. Interdisciplinary Coordination

A dynamic analysis requires a team of engineering geologists, seismologists, and structural engineers. They must work together in an integrated approach so that elements of conservatism are not unduly compounded. An example of undue conservatism includes using a rare event as the MCE, upper bound values for the PGA, upper bound values for the design response spectra, and conservative criteria for determining the earthquake resistance of the structure. The steps in performing a dynamic analysis should be fully coordinated to develop a reasonably conservative design with respect to the associated risks. The structural engineers responsible for the dynamic structural analysis should be actively involved in the process of characterizing the earthquake ground motions (see

paragraph 5-6) in the form required for the methods of dynamic analysis to be used.

5-5. Performance Criteria for Response to Site-Dependent Earthquakes

a. Maximum credible earthquake. Gravity dams should be capable of surviving the controlling MCE without a catastrophic failure that would result in loss of life or significant damage to property. Inelastic behavior with associated damage is permissible under the MCE.

b. Operating basis earthquake. Gravity dams should be capable of resisting the controlling OBE within the elastic range, remain operational, and not require extensive repairs.

5-6. Geological and Seismological Investigation

A geological and seismological investigation of all damsites is required for projects located in seismic zones 2 through 4. The objectives of the investigation are to establish controlling maximum and credible operating basis earthquakes and the corresponding ground motions for each and to assess the possibility of earthquake-induced foundation dislocation at the site. Selecting the controlling earthquakes is discussed below. Additional information is also available in TM 5-809-10-1.

5-7. Selecting the Controlling Earthquakes

a. Maximum credible earthquake. The first step for selecting the controlling MCE is to specify the magnitude and/or modified Mercalli (MM) intensity of the MCE for each seismotectonic structure or source area within the region examined around the site. The second step is to select the controlling MCE based on the most severe vibratory ground motion within the predominant frequency range of the dam and determine the foundation dislocation, if any, capable of being produced at the site by the candidate MCE's. If more than one candidate MCE produce the largest ground motions in different frequency bands significant to the response of the dam, each should be considered a controlling MCE.

b. Operating basis earthquake.

(1) The selection of the OBE is based upon the desired level of protection for the project from earthquake-induced damage and loss of service project life. The project life of new dams is usually taken as 100 years. The probability of exceedance of the OBE

during the project life should be no greater than 50 percent unless the cost savings in designing for a less severe earthquake outweighs the risk of incurring the cost of repairs and loss of service because of a more severe earthquake.

(2) The probabilistic analysis for the OBE involves developing a magnitude frequency or epicentral intensity frequency (recurrence) relationship of each seismic source; projecting the recurrence information from regional and past data into forecasts concerning future occurrence; attenuating the severity parameter, usually either PGA of MM intensity, to the site; determining the controlling recurrence relationship for the site; and finally, selecting the design level of earthquake based upon the probability of exceedance and the project life.

5-8. Characterizing Ground Motions

a. General. After specifying the location and magnitude (or epicentral intensity) of each candidate earthquake and an appropriate regional attenuation relationship, the characteristics of vibratory ground motion expected at the site can be determined. Vibratory ground motions have been described in a variety of ways, such as peak ground motion parameters, acceleration-time records (accelerograms), or response spectra (Hayes 1980, and Krinitzsky and Marcuson 1983). For the analysis and design of concrete dams, the controlling characterization of vibratory ground motion should be a site-dependent design response spectra.

b. Site-specific design response spectra.

(1) Wherever possible, site-specific design response spectra should be developed statistically from response spectra of strong motion records of earthquakes that have similar source and propagation path properties as the controlling earthquake(s) and are recorded on a foundation similar to that of the dam. Important source properties include magnitude and, if possible, fault type and tectonic environment. Propagation path properties include distance, depth, and attenuation. As many accelerograms as possible that are recorded under comparable conditions and have a predominant frequency similar to that selected for the design earthquake should be included in the development of the design response spectra. Also, accelerograms should be selected that have been corrected for the true baseline of zero acceleration, for errors in digitization, and for other irregularities (Schiff and Bogdanoff 1967).

(2) Where a large enough ensemble of site-specific strong motion records is not available, design response spectra may be approximated by scaling that ensemble of records that represents the best estimate of source, propagation path, and site properties. Scaling factors can be obtained in several ways. The scaling factor may be determined by dividing the peak or effective peak acceleration specified for the controlling earthquake by the peak acceleration of the record being rescaled. The peak velocity of the record should fall within the range of peak velocities specified for the controlling earthquake, or the record should not be used. Spectrum intensity can be used for scaling by using the ratio of the spectrum intensity determined for the site and the spectrum intensity of the record being rescaled (USBR 1978). Acceleration attenuation relationships can be used for scaling by dividing the acceleration that corresponds to the source distance and magnitude of the controlling earthquake by the acceleration that corresponds to the source distance and magnitude of the record being rescaled (Guzman and Jennings 1970). Because the scaling of accelerograms is an approximate operation at best, the closer the characteristics of the actual earthquake are to those of the controlling earthquake, the more reliable the results. For this reason, the scaling factor should be held to within a range of 0.33 to 3 for gravity dam.

(3) Guidance for developing design response spectra, statistically, from strong motion records is given in Vanmarcke (1979).

(4) Site-dependent response spectra developed from strong motion records, as described in paragraphs 5-8*b*, should have amplitudes equal to or greater than the mean response spectrum for the appropriate foundation given by Seed, Ugas, and Lysmer (1976), anchored by the PGA determined for the site. This minimum response spectrum may be anchored by an effective PGA determined for the site, but supporting documentation for determining the effective PGA will be required (Newmark and Hall 1982).

(5) A mean smooth response spectrum of the response spectra of records chosen should be presented for each damping value of interest. The statistical level of response spectra used should be justified based on the degree of conservatism in the preceding steps of the seismic design process and the thoroughness of the development of the design response spectra. If a rare event is used as the controlling earthquake and the earthquake records are scaled by upper bound values of ground motions, then use a response spectrum corresponding to

the mean of the amplification factors if the response spectrum is based on five or more earthquake records.

c. Accelerograms for acceleration-time history analysis. Accelerograms used for dynamic input should be compatible with the design response spectrum and account for the peak ground motions parameters, spectrum intensity, and duration of shaking. Compatibility is defined as the envelope of all response spectra derived from the selected accelerograms that lie below the smooth design response spectrum throughout the frequency range of structural significance.

5-9. Dynamic Methods of Stress Analysis

a. General. A dynamic analysis determines the structural response based on the characteristics of the structure and the nature of the earthquake loading. Dynamic methods usually employ the modal analysis technique. This technique is based on the simplifying assumption that the response in each natural mode of vibration can be computed independently and the modal responses can be combined to determine the total response (Chopra 1987). Modal techniques that can be used for gravity dams include a simplified response spectrum method and finite element methods using either a response spectrum or acceleration-time records for the dynamic input. A dynamic analysis should begin with the response spectrum method and progress to more refined methods if needed. A time-history analysis is used when yielding (cracking) of the dam is indicated by a response spectrum analysis. The time-history analysis allows the designer to determine the number of cycles of nonlinear behavior, the magnitude of excursion into the nonlinear range, and the time the structure remains nonlinear.

b. Simplified response spectrum method.

(1) The simplified response spectrum method computes the maximum linear response of a nonoverflow section in its fundamental mode of vibration due to the horizontal component of ground motion (Chopra 1987). The dam is modeled as an elastic mass fully restrained on a rigid foundation. Hydrodynamic effects are modeled as an added mass of water moving with the dam. The amount of the added water mass depends on the fundamental frequency of vibration and mode shape of the dam and the effects of interaction between the dam and reservoir. Earthquake loading is computed directly from the spectral acceleration, obtained from the design earthquake response spectrum, and the dynamic properties of the structural system.

(2) This simplified method can be used also for an ungated spillway monolith that has a section similar to a nonoverflow monolith. A simplified method for gated spillway monoliths is presented in WES Technical Report SL-89-4 (Chopra and Tan 1989).

(3) The program SDOFDAM is available to easily model a dam using the finite element method and Chopra's simplified procedure for estimating the hydrodynamic loading. This analysis provides a reasonable first estimate of the tensile stress in the dam. From that estimate, one can decide if the design is adequate or if a refined analysis is needed.

c. Finite element methods.

(1) General. The finite element method is capable of modeling the horizontal and vertical structural deformations and the exterior and interior concrete, and it includes the response of the higher modes of vibrations, the interaction effects of the foundation and any surrounding soil, and the horizontal and vertical components of ground motion.

(2) Finite element response spectrum method.

(a) The finite element response spectrum method can model the dynamic response of linear two- and three-dimensional structures. The hydrodynamic effects are modeled as an added mass of water moving with the dam using Westergaard's formula (Westergaard 1933). The foundations are modeled as discrete elements or a half space.

(b) Six general purpose finite element programs are compared by Hall and Radhakrishnan (1983).

(c) A finite element program computes the natural frequencies of vibration and corresponding mode shapes for specified modes. The earthquake loading is computed from earthquake response spectra for each mode of vibration induced by the horizontal and vertical components of ground motion. These modal responses are combined to obtain an estimate of the maximum total response. Stresses are computed by a static analysis of the dam using the earthquake loading as an equivalent static load.

(d) The complete quadratic combination (CQC) method (Der Kiureghian 1979 and 1980) should be used to combine the modal responses. The CQC method degenerates to the square root of the sum of squares (SRSS) method for two-dimensional structures in which

the frequencies are well separated. Combining modal maxima by the SRSS method can dramatically overestimate or significantly underestimate the dynamic response for three-dimensional structures.

(e) The finite element response spectrum method should be used for dam monoliths that cannot be modeled two dimensionally or if the maximum tensile stress from the simplified response spectrum method (paragraph 5-9*b*) exceeds 15 percent of the unconfined compressive strength of the concrete.

(f) Normal stresses should be used for evaluating the results obtained from a finite element response spectrum analysis. Finite element programs calculate normal stresses that, in turn, are used to compute principal stresses. The absolute values of the dynamic response at different time intervals are used to combine the modal responses. These calculations of principal stress overestimate the actual condition. Principal stresses should be calculated using the finite element acceleration-time history analysis for a specific time interval.

(3) Finite element acceleration-time history method.

(a) The acceleration-time history method requires a general purpose finite element program or the special purpose computer program called EADHI. EADHI can model static and dynamic responses of linear two-dimensional dams. The hydrodynamic effects are modeled using the wave equation. The compressibility of water and structural deformation effects are included in computing the hydrodynamic pressures. EADHI was developed assuming a fixed base for the dam. The most comprehensive two-dimensional earthquake analysis program available for gravity dams is EAGD84, which can model static and dynamic responses of linear two-dimensional dams, including hydrodynamic and foundation interaction. Dynamic input for EADHI and EAGD84 is an acceleration time record.

(b) The acceleration-time history method computes the natural frequencies of vibration and corresponding mode shapes for specified modes. The response of each mode, in the form of equivalent lateral loads, is calculated for the entire duration of the earthquake acceleration-time record starting with initial conditions, taking a small time interval, and computing the response at the end of each time interval. The modal responses are added for each time interval to yield the total response. The stresses are computed by a static analysis for each time interval.

(c) An acceleration-time history analysis is appropriate if the variation of stresses with time is required to evaluate the extent and duration of a highly stressed condition.

Chapter 6
Temperature Control of Mass Concrete

6-1. Introduction

Temperature control of mass concrete is necessary to prevent cracking caused by excessive tensile strains that result from differential cooling of the concrete. The concrete is heated by reaction of cement with water and can gain additional heat from exposure to the ambient conditions. Cracking can be controlled by methods that limit the peak temperature to a safe level, so the tensile strains developed as the concrete cools to equilibrium are less than the tensile strain capacity.

6-2. Thermal Properties of Concrete

a. General. The properties of concrete used in thermal studies for the design of gravity dams are thermal diffusivity, thermal conductivity, specific heat, coefficient of thermal expansion, heat of hydration of the cement, tensile strain capacity, and modulus of elasticity. The most significant factor affecting the thermal properties is the composition of the aggregates. The selection of suitable aggregates is based on other considerations, so little or no control can be exercised over the thermal properties of the aggregates. Type II cement with optional low heat of hydration limitation and a cement replacement are normally specified. Type IV low-heat cement has not been used in recent years, because in most cases heat development can be controlled by other measures and type IV cement is not generally available.

b. Thermal conductivity. The thermal conductivity of a material is the rate at which it transmits heat and is defined as the ratio of the flux of heat to the temperature gradient. Water content, density, and temperature significantly influence the thermal conductivity of a specific concrete. Typical values are 2.3, 1.7, and 1.2 British thermal units (Btu)/hour/foot/Fahrenheit degree (°F) for concrete with quartzite, limestone, and basalt aggregates, respectively.

c. Thermal diffusivity. Diffusivity is described as an index of the ease or difficulty with which concrete undergoes temperature change and, numerically, is the thermal conductivity divided by the product of specific heat and density. Typical diffusivity values for concrete range from 0.03 square foot/hour for basalt concrete to 0.06 square foot/hour for quartzite concrete.

d. Specific heat. Specific heat or heat capacity is the heat required to raise a unit weight of material 1 degree. Values for various types of concrete are about the same and vary from 0.22 to 0.25 Btu's/pound/°F.

e. Coefficient of thermal expansion. The coefficient of thermal expansion can be defined as the change in linear dimension per unit length divided by the temperature change expressed in millionths per °F. Basalt and limestone concretes have values from 3 to 5 millionths/°F; quartzite concretes range up to 8 millionths/°F.

f. Heat of hydration. The reaction of water with cement is exothermic and generates a considerable amount of heat over an extended period of time. Heats of hydration for various cements vary from 60 to 95 calories/gram at 7 days and 70 to 110 calories/gram at 28 days.

g. Tensile strain capacity. Design is based on maximum tensile strain. The modulus of rupture test (CRD-C 16) is done on concrete beams tested to failure under third-point loading. Tensile strain capacity is determined by dividing the modulus of rupture by the modulus of elasticity. Typical values range from 50 to 200 millions depending on loading rate and type of concrete.

h. Creep. Creep of concrete is deformation that occurs while concrete is under sustained stress. Specific creep is creep under unit stress. Specific creep of mass concrete is in the range of 1.4×10^{-6}/pounds per square inch (psi).

i. Modulus of elasticity. The instantaneous loading modulus of elasticity for mass concrete ranges from about 1.5 to 6×10^6 psi and under sustained loading from about 0.5 to 4×10^6 psi.

6-3. Thermal Studies

a. General. During the design of gravity dams, it is necessary to assess the possibility that strain induced by temperature changes in the concrete will not exceed the strain capacity of the concrete. Detailed design procedures for control of the generation of heat and volume changes to minimize cracking may be found in the *ACI Manual of Concrete Practice*, Section 207. The following concrete parameters should be determined by a division laboratory: heat of hydration (CRD-C 229), adiabatic temperature rise (CRD-C 38), thermal conductivity (CRD-C 44), thermal diffusivity (CRD-C 37), specific

heat (CRD-C 124), coefficient of thermal expansion (CRD-C 397, 125, and 126), creep (CRD-C 54), and tensile strain capacity (CRD-C 71). Thermal properties testing should not be initiated until aggregate investigations have proceeded to the point that the most likely aggregate sources are determined and the availability of cementitious material is known.

b. Allowable peak temperature. The peak temperature for the interior mass concrete must be controlled to prevent cracking induced by surface contraction. The allowable peak temperature commonly used to prevent serious cracking in mass concrete structures is the mean annual ambient temperature plus the number of degrees Fahrenheit determined by dividing the tensile strain capacity by the coefficient of linear expansion. This assumes that the concrete will be subjected to 100-percent restraint against contraction. When the potential temperature rise of mass concrete is reduced to this level, the temperature drop that causes tensile strain and cracking is reduced to an acceptable level.

6-4. Temperature Control Methods

The temperature control methods available for consideration all have the basic objective of reducing increases in temperature due to heat of hydration, reducing thermal differentials within the structure, and reducing exposure to cold air at the concrete surfaces that would create cracking. The most common techniques are the control of lift thickness, time interval between lifts, maximum allowable placing temperature of the concrete, and surface insulation. Postcooling may be economical for large structures. Analysis should be made to determine the most economic method to restrict temperature increases and subsequent temperature drops to levels just safely below values that could cause undesirable cracking. For structures of limited complexity, such as conventionally shaped gravity dams, satisfactory results may be obtained by use of the design procedures in ACI 207 "Mass Concrete for Dams and Other Massive Structures." Roller compacted concrete thermal control options include the installation of contraction joints, winter construction, mixture design, and increased heat dissipation. Contraction joints can be created by inserting a series of cuts or metal plates into each lift to produce a continuous vertical joint. Using very high production and placement rates, RCC construction can be limited to colder winter months without excessive schedule delays. The normal lift height of 1 to 2 feet provides for an increased rate of heat dissipation during cool weather.

Chapter 7
Structural Design Considerations

7-1. Introduction

This chapter discusses the layout, design, and construction considerations associated with concrete gravity dams. These general considerations include contraction and construction joints, waterstops, spillways, outlet works, and galleries. Similar considerations related to RCC gravity dams are addressed in Chapter 9.

7-2. Contraction and Construction Joints

a. To control the formation of cracks in mass concrete, vertical transverse contraction (monolith) joints will generally be spaced uniformly across the axis of the dam about 50 feet apart. Where a powerhouse forms an integral part of a dam and the spacing of the units is in excess of this dimension, it will be necessary to increase the joint spacing in the intake block to match the spacing of the joints in the powerhouse. In the spillway section, gate and pier size and other requirements are factors in the determination of the spacing of the contraction joints. The location and spacing of contraction joints should be governed by the physical features of the damsite, details of the appurtenant structures, results of temperature studies, placement rates and methods, and the probable concrete mixing plant capacity. Abrupt discontinuities along the dam profile, material changes, defects in the foundation, and the location of features such as outlet works and penstock will also influence joint location. In addition, the results of thermal studies will provide limitations on monolith joint spacing for assurance against cracking from excessive temperature-induced strains. The joints are vertical and normal to the axis, and they extend continuously through the dam section. The joints are constructed so that bonding does not exist between adjacent monoliths to assure freedom of volumetric change of individual monoliths. Reinforcing should not extend through a contraction joint. At the dam faces, the joints are chamfered above minimum pool level for appearance and for minimizing spalling. The monoliths are numbered, generally sequentially, from the right abutment.

b. Horizontal or nearly horizontal construction joints (lift joints) will be spaced to divide the structure into convenient working units and to control construction procedure for the purpose of regulating temperature changes. A typical lift will usually be 5 feet consisting of three 20-inch layers, or 7-1/2 feet consisting of five 18-inch layers. Where necessary as a temperature control

measure, lift thickness may be limited to 2-1/2 feet in certain areas of the dam. The best lift height for each project will be determined from concrete production capabilities and placing methods. EM 1110-2-2000 provides guidance on establishing lift thickness.

7-3. Waterstops

A double line of waterstops should be provided near the upstream face at all contraction joints. The waterstops should be grouted 18 to 24 inches into the foundation or sealed to the cutoff system and should terminate near the top of the dam. For gated spillway sections, the tops of the waterstops should terminate near the crest of the ogee. A 6- to 8-inch-diameter formed drain will generally be provided between the two waterstops. In the nonoverflow monolith joints, the drains extend from maximum pool elevation and terminate at about the level of, and drain into, the gutter in the grouting and drainage gallery. In the spillway monolith joints, the drains extend from the gate sill to the gallery. A single line of waterstops should be placed around all galleries and other openings crossing monolith joints. EM 1110-2-2102 provides further details and guidance for the selection and use of waterstops and other joint materials.

7-4. Spillway

a. The primary function of a spillway is to release surplus water from reservoirs and to safely bypass the design flood downstream in order to prevent overtopping and possible failure of the dam. Spillways are classified as controlled (gate) or uncontrolled (ungated). The overflow (ogee) spillway is the type usually associated with concrete gravity dams. Other less common spillway types such as chute, side channel, morning glory, and tunnel are not addressed in this manual.

b. An overflow spillway profile is governed in its upper portions by hydraulic considerations rather than by stability requirements. The downstream face of the spillway section terminates either in a stilling basin apron or in a bucket type energy dissipator, depending largely upon the nature of the site and upon the tailwater conditions. The design of the spillway shall include the stability and internal stress analysis and the structural performance. Loadings should be consistent with those discussed in Chapter 4. Operating equipment should be designed to be operational following a maximum credible earthquake.

c. Discharge over the spillway or flip bucket section must be confined by sidewalls on either side, terminating in training walls extending along each side of the stilling

basin or flip bucket. Height and length of training walls are usually determined by model tests or from previous tests of similar structures. Sidewalls should be of sufficient height to contain the spillway design flow, with a 2-foot freeboard. Negative pressures (see EM 1110-2-1603) due to flowing water should be considered in the design of the sidewalls, with the maximum allowance (see EM 1110-2-2400) being made at the stilling basin, decreasing uniformly to no allowance at the crest. Sidewalls are usually designed as cantilevers projecting out of the monolith. A wind load of 30 pounds/square foot or earthquake loading should be assumed for design of reinforcing in the outer face of the walls. The spillway section surfaces should be designed to withstand the high flow velocities expected during peak discharge and reduced pressures resulting from the hydrodynamic effects.

d. The dynamic loads occurring in the energy dissipators will include direct impact, pulsating loads from turbulence, multidirectional and deflected hydraulic flows, surface erosion from high velocities and debris, and cavitation. The downstream end of the dissipator should include adequate protection against undermining from turbulence and eddies. Concrete apron, riprap, or other measures have been used for stabilization.

7-5. Spillway Bridge

a. Bridges are provided across dam spillways to furnish a means of access for pedestrian and vehicular traffic between the nonoverflow sections; to provide access or support for the operating machinery for the crest gates; or, usually, to serve both purposes. In the case of an uncontrolled spillway and in the absence of vehicular traffic, access between the nonoverflow sections may be provided by a small access bridge or by stair shafts and a gallery beneath the spillway crest.

b. The design of a deck-type, multiple-span spillway bridge should generally conform to the following criteria. The class of highway design loading will normally not be less than HS-20. Special loadings required for performing operation and maintenance functions and those that the bridge is subjected to during construction should be taken into account, including provisions for any heavy concentrated loads. Heavy loadings for consideration should include those due to powerhouse equipment transported during construction, mobile cranes used for maintenance, and gantry cranes used to operate the regulating outlet works and to install spillway stoplogs. If the structure carries a state or county highway, the design will usually conform to the standard specification for highway bridges adopted by the American Association of State Highway and Transportation Officials (AASHTO).

c. Materials used in the design and construction of the bridge should be selected on the basis of life cycle costs and functional requirements. Floors, curbs, and parapets should be reinforced concrete. Beams and girders may be structural steel, precast or cast-in-place reinforced concrete, or prestressed concrete. Prestressed concrete is often used because it combines economy, simple erection procedures, and low maintenance.

7-6. Spillway Piers

a. For uncontrolled spillways, the piers function as supports for the bridge. On controlled spillways, the piers will also contain the anchorage or slots for the crest gates and may support fixed hoists for the gates. The piers are generally located in the middle of the monolith, and the width of pier is usually determined by the size of the gates, with the average width being between 8 and 10 feet. The spillway piers in RCC dams are constructed with conventional concrete.

b. Since each pier supports a gate on each side, the following pier loading conditions should be investigated:

(1) Case 1--both gates closed and water at the top of gates.

(2) Case 2--one gate closed and the other gate wide open with water at the top of the closed gate.

(3) Case 3--one gate closed and the other open with bulkheads in place and water at the top of the closed gate.

c. Cases 1 and 3 result in maximum horizontal shear normal to the axis of the dam and the largest overturning moment in the downstream direction.

d. Case 2 results in lower horizontal shear and downstream overturning moment, but in addition the pier will have a lateral bending moment due to the water flowing through the open gate and to the hoisting machinery when lifting a closed gate. A torsional shear in the horizontal plane will also be introduced by the reaction of the closed gate acting on one side of the pier. When tainter gates with inclined end frames are used, Cases 2 and 3 introduce the condition of the lateral component of the thrust on the trunion as a load on only one side of the pier in addition to the applicable loads indicated above.

7-7. Outlet Works

a. The outlet works for concrete dams are usually conduits or sluices through the mass with an intake structure on the upstream face, gates or valves for regulation control, and an energy dissipator on the downstream face. Multiple conduits are normally provided because of economics and operating flexibility in controlling a wide range of releases. The conduits are frequently located in the center line of the overflow monoliths and discharge into the spillway stilling basin. Outlet works located in nonoverflow monoliths will require a separate energy dissipator. All conduits may be at low level, or some may be located at one or more higher levels to reduce the head on the gates, to allow for future reservoir silting, or to control downstream water quality and temperature. The layout, size, and shape of the outlet works are based on hydraulic and hydrology requirements, regulation plans, economics, site conditions, operation and maintenance needs, and interrelationship to the construction plan and other appurtenant structures. Conduits may be provided for reservoir evacuation, regulation of flows for flood control, emergency drawdown, navigation, environmental (fish), irrigation, water supply, maintaining minimum downstream flows and water quality, or for multiple purposes. Low-level conduits are used to aid water quality reservoir evacuation and are sometimes desirable for passage of sediment. These openings are generally unlined except for short sections adjacent to the control gates. For lined conduits, it is assumed that the liner is designed for the full loading. In conduits where velocities will be 40 feet/second or higher, precautions will be taken to ensure that the concrete in the sidewalls and inverts will be of superior quality. If the dam includes a power intake section, penstocks will be provided and designed in accordance with EM 1110-2-3001.

b. The effect of project functions upon outlet works design and hydraulic design features, including trashrack design and types for sluice outlets, are discussed in EM 1110-2-1602. A discussion of the structural features of design for penstocks and trashracks for power plant intakes is included in EM 1110-2-3001. The structural design of outlet works is addressed in EM 1110-2-2400.

7-8. Foundation Grouting and Drainage

It is good engineering practice to grout and drain the foundation rock of gravity dams. A well-planned and executed grouting program should assist in disclosing weaknesses in the foundation and improving any existing defects. The program should include area grouting for foundation treatment and curtain grouting near the upstream face for seepage cutoff through the foundation. Area grouting is generally done before concrete placement. Curtain grouting is commonly done after concrete has been placed to a considerable height or even after the structure has been completed. A line of drainage holes is drilled a few feet downstream from the grout curtain to collect seepage and reduce uplift across the base. Detailed information on technical criteria and guidance on foundation grouting is contained in EM 1110-2-3506.

7-9. Galleries

A system of galleries, adits, chambers, and shafts is usually provided within the body of the dam to furnish means of access and space for drilling and grouting and for installation, operation, and maintenance of the accessories and the utilities in the dam. The primary considerations in the arrangement of the required openings within the dam are their functional usefulness and efficiency and their location with respect to maintaining the structural integrity.

a. Grouting and drainage gallery. A gallery for grouting the foundation cutoff will extend the full length of the dam. It will also serve as a collection main for seepage from foundation drainage holes and the interior drainage holes. The location of the gallery should be near the upstream face and as near the rock surface as feasible to provide the maximum reduction in overall uplift. A minimum distance of 5 feet should be maintained between the foundation surface and gallery floor and between the upstream face and the gallery upstream wall. It has been standard practice to provide grouting galleries 5 feet wide by 7 feet high. Experience indicates that these dimensions should be increased to facilitate drilling and grouting operations. Where practicable, the width should be increased to 6 or 8 feet and the height to 8 feet. A gutter may be located along the upstream wall of the gallery where the line of grout holes is situated to carry away drill water and cuttings. A gutter should be locatedalong the downstream gallery wall to carry away flows from the drain pipes. The gallery is usually arranged as a series of horizontal runs and stair flights. The stairs should be provided with safety treads or a nonslip aggregate finish. Metal treads are preferable where it is probable that equipment will be skidded up or down the steps since they provide protection against chipping of concrete. Where practicable, the width of tread and height of riser should be uniform throughout all flights of stairs and should never change in any one flight. Further details on the grouting and drainage gallery are covered in EM 1110-2-3506.

b. Gate chambers and access galleries. Gate chambers are located directly over the service and emergency sluice gates. These chambers should be sized to accommodate the gate hoists along with related mechanical and electrical equipment and should provide adequate clearances for maintenance. Access galleries should be sufficient size to permit passage of the largest component of the gates and hoists and equipment required for maintenance. Drainage gutters should be provided and the floor of the gallery sloped to the gutter with about 1/4 inch/foot slope.

7-10. Instrumentation

Structural behavior instrumentation programs are provided for concrete gravity dams to measure the structural integrity of the structure, check design assumptions, and monitor the behavior of the foundation and dam during construction and the various operating phases. The extent of instrumentation at projects will vary between projects depending on particular site conditions, the size of the dam, and needs for monitoring critical sections. Instrumentation can be grouped into those that either directly or indirectly measure conditions related to the safety of the structure. Plumbing, alignment, uplift, and seismic instruments fall into the category of safety instruments. In the other group, the instruments measure quantities such as stress and strain, length change, pore pressure, leakage, and temperature change. Details and guidance on the planning of instrumentation programs, types of instruments, and the preparation, installation, and collection of data are provided in EM 1110-2-4300.

Chapter 8
Reevaluation of Existing Dams

8-1. General

Existing gravity dams and foundations should be reevaluated for integrity, strength, and stability when:

a. It is evident that distress has occurred because of an accident, aging, or deterioration.

b. Design criteria have become more stringent.

c. Excavation is to be performed near existing structures.

d. Structural deficiencies have been detected.

e. Actual loadings are, or anticipated loadings will be, greater than those used in the original design. Loadings can increase as a result of changed operational procedures or operational deficiencies, an increase in dam height, or an increase in the maximum credible earthquake as a result of seismological investigations. Conditions such as excessive uplift pressures, unusual horizontal or vertical displacements, increased seepage through the concrete or foundation, and structural cracking are indications that a reevaluation should be performed.

8-2. Reevaluation

The reevaluation should be based on current design criteria and prevailing geological, structural, and hydrological conditions. If the investigations indicate a fundamental deficiency, then the initial effort should concentrate on restoring the dam to a safe and acceptable operating condition. Efforts could include measures to reduce excessive uplift pressures, reduce leak, repair cracks, or restore deteriorated concrete. Should restoration costs be unreasonable or should the fundamental deficiency be due to changes in load or stability criteria, a detailed analysis should be performed in accordance with the following procedures. The evaluation and repair of concrete structures is covered by EM 1110-2-2002. Reevaluation of structures not designed to current standards should be in accordance with the requirements of ER 1110-2-100.

8-3. Procedures

The following procedures shall be used in evaluating current structural conditions and determining the necessary measures for rehabilitation of existing concrete gravity dams.

a. Existing data. Collect and review all the available information for the structure including geologic and foundation data, design drawings, as-built drawings, periodic inspection reports, damage reports, repair and maintenance records, plans of previous modifications to the structure, measurements of movement, instrumentation data, and other pertinent information. Any unusual structural behavior that may be an indication of an unsafe condition or any factor that may contribute to the weakening of the structure's stability should be noted and investigated further.

b. Site inspection. Inspect and examine the existing structure and site conditions. Any significant difference in structure details and loading conditions between existing conditions and design plans and any major damage due to erosion, cavitation, undermining, corrosion, cracking, chemical reaction, or general deterioration should be identified and evaluated.

c. Preliminary analyses. Perform the preliminary analyses based on current structural criteria and available data. If the structure does not meet the current criteria, list the possible remedial schemes and prepare a preliminary cost estimate for each scheme. ER 1130-2-417 should be followed as applicable.

d. Design meeting. Schedule a meeting when the preliminary analyses indicate that the structure does not meet current criteria. The meeting should include representatives from the District, Division, CECW-E, and CECW-O to decide on a plan for proposed analyses, the extent of the sampling and testing program, the remedial schemes to be studied, and the proposed schedule. This meeting will facilitate the design effort and should obviate the need for major revisions or additional studies when the results are submitted for review and approval.

e. Parametric study. Perform a parametric study to determine the effect of each parameter on the structure's safety. The parameters to be studied should include, but not be limited to, unit weight of concrete, groundwater levels, uplift pressures, and shear strength parameters of rock fill material, rock foundation, and structure-foundation interface. The maximum variation of each parameter should be considered in determining its effect.

f. Field investigations. Develop an exploration, sampling, testing, and instrumentation program, if needed, to

determine the magnitude and reasonable range of variation for the parameters that have significant effects on the safety of the structure as determined by the parametric study. The Division Material Laboratory should be used to the maximum extent practicable to perform the testing in accordance with ER 1110-1-8100.

g. Detailed structural analyses. Perform detailed analyses using data obtained from studies, field investigations, and procedures outlined in Chapters 4 and 5. Three-dimensional modeling should be used as appropriate to more accurately predict the structural behavior.

h. Refined structural analysis. The conventional methods described in Chapters 4 and 5 may be more conservative than necessary, especially when making a determination as to the need for remedial strengthening to improve the stability of an existing dam. If the conventional analyses indicate remedial strengthening is required, then a refined finite element analysis should be performed. This refined analysis should accurately model the strength and stiffness of the dam and foundation to determine the following:

(1) The extent of tensile cracking at the dam foundation interface.

(2) The base area in compression.

(3) The actual magnitude and distribution of foundation pressures.

(4) The magnitude and distribution of concrete stresses.

Information relative to refined stability analysis procedures can be found in Technical Report REMR-CS-120 (Eberling et al., in preparation).

i. Review and approval. Present the results of detailed structural analyses and cost estimates for remedial measures to the Division Office for review and approval. If a deviation from current structural criteria was made in the analyses, the results should be forwarded to CECW-ED for approval. The required basis for deviating from current structural criteria is given in paragraph 8-4*b*.

j. Plans and specifications. Develop design plans, specifications, and a cost estimate for proposed remedial measures in accordance with ER 1110-2-1200.

8-4. Considerations of Deviation from Structural Criteria

a. The purpose of incorporating a factor of safety in structural design is to provide a reserve capacity with respect to failure and to account for strength variability of the dam and foundation materials. The required margin depends on the consequences of failure and on the degree of uncertainties due to loading variations, analysis simplifications, design assumptions, variations in material strengths, variations in construction control, and other factors. For evaluation of existing structures, a higher degree of confidence may be achieved when the critical parameters can be determined accurately at the site. Therefore, deviation from the current structural criteria for an existing structure may be allowed under the conditions listed in paragraph 8-4*b*.

b. In addition to the detailed analyses and cost estimates as listed in paragraph 8-3*h*, the following information should also be presented with the request for a deviation from the current structural criteria:

(1) Past performance of the structure, including instrumentation data and a description of the structure condition such as cracking, spalling, displacements, etc.

(2) The anticipated remaining life of the structure.

(3) A description of consequences in case of failure.

c. Approval of the deviation depends upon the degree of confidence in the accuracy of design parameters determined in the field, the remaining life of the structure, and the potential adverse effect on lives, property, and services in case of failure.

8-5. Structural Requirements for Remedial Measure

When it is determined that remedial measures are required for the existing structure, they should be designed to meet the structural criteria of Chapter 4.

8-6. Methods of Improving Stability in Existing Structures

a. General. Several methods are available for improving the rotational and sliding stability of concrete gravity dams. In general, the methods can be categorized

as those that reduce loadings, in particular uplift, or those that add stabilizing forces to the structure and increase overturning or shear-frictional resistance. Stressed foundation anchor systems are considered one of the most economical methods of increasing rotational and sliding resistance along the base of the dam. Foundation grouting and drainage may also be effective in reducing uplift, reducing foundation settlements and displacements, thereby increasing bearing capacity. Regrouting the foundation could adversely affect existing foundation drainage systems unless measures are taken to prevent plugging the drains; otherwise, drain redrilling will be required. Various methods of transferring load to more competent adjacent structures or foundation material through shear keys, buttresses, underpinning, etc., are also possible ways of improving stability.

b. Reducing uplift forces. In many instances, measured uplift pressures are substantially less than those used in the original design. These criteria limit drain efficiency to a maximum of 50 percent. Many designs are based on efficiencies less than 50 percent. Existing drainage systems can produce efficiencies of 75 percent or more if they extend through the most pervious layers of the foundation, if the elevation of the drainage gallery is at or near tailwater, and if the drains are closely spaced and effectively maintained. If measured uplift pressures are substantially less than design values, then parametric studies should determine what benefit it may have towards improving stability. Uplift pressures less than design allowables should be data from reliable instrumentation which assures that the measured uplift is indicative of pressures within the upper zones and along the entire foundation. Uplift pressures can be reduced by additional foundation grouting and re-establishing drains. Uplift may also be reduced by increasing the depth of existing drains, adding new drains, or rehabilitating existing drains by reaming and cleaning.

c. Prestressed anchors. Prestressed anchors with double corrosion protection may be used to stabilize existing concrete monoliths, but generally should not be used in the design of new concrete gravity dams. They are effective in improving sliding resistance, resultant location, and excessive foundation pressure. Anchors may be used to secure thrust blocks or stilling basins for the sole purpose of improving sliding stability. The anchor force required to stabilize a dam will depend largely on the orientation of the anchors. Anchors should be oriented for maximum efficiency subject to constraints of access, embedded features, galleries, and stress concentrations they induce in the dam. Analyses of tensile stresses under anchor heads should be made, and reinforcing should be provided as required. Tendon size, spacing, and embedment length should be based on the required anchor force, and should be provided the geotechnical engineer for determination of the required embedment length. Design, installation, and testing of anchors and anchorages should be guided by information in "Recommendations for Prestressed Rock and Soil Anchors" (Post-Tensioning Institute (PTI) 1985). Allowable bond stresses used to determine the length of embedment between grout and rocks are recommended to be one half of the ultimate bond stress determined by tests. The typical values of bond strength given in the above referenced PTI publication may be used in lieu of test values during design, but the design value should be verified by test before or during construction. The first three anchors installed and a minimum of 2 percent of the remaining anchors selected by the engineer should be performance tested. All other anchors must be proof tested upon installation in accordance with the PTI recommendations. Additionally, initial lift-off readings should be taken after the anchor is seated and before the jack is removed. Lift-off tests of random anchors selected by the engineer should be made 7 days after lock-off and prior to secondary grouting. Long-term monitoring of selected anchors using load cells and unbonded tendons should be employed where unusual conditions exist or the effort and expense can be justified by the importance of the structure. In addition to stability along the base of the dam, prestressed anchors may be required for deep-seated stability problems as discussed in the following paragraph. Non-prestressed anchors shall not be used to improve the stability of dams.

8-7. Stability on Deep-Seated Failure Planes

A knowledge of the rock structure of a foundation is crucial to a realistic stability analysis on deep-seated planes. If instability is to occur, it will take place along zones of weakness within the rock mass. A team effort between the geotechnical and structural engineers is important in evaluating the foundation and its significance to the design of the dam. Deep-seated sliding is of primary interest as it is the most common problem encountered. Significant foundation features are: rock surface joint patterns that admit water to potential deep-seated sliding planes; inclination of joints and fracturing that affect passive resistance; relative permeability of foundation materials that affect uplift; and discontinuities such as gouge zones and faulting which affect both strength and uplift along failure planes. Strength values for failure planes are required for design. As these values are often difficult to define with a high level of confidence, they should be described in terms of expected values and standard deviations. Analyses of resultant location and

maximum bearing pressure will also be required. Criteria for these loading conditions will be the same as in Chapter 4 for the dam.

a. Method and assumptions. Stability on deep-seated planes is similar to methods described in Chapter 4 for the dam. Tensile strength within the foundation is neglected except where it can be demonstrated by exploration and testing. Vertical and near vertical joints are assumed to be fully pressurized by the pool to which they are exposed. Normally a pressurized vertical joint will be assumed to exist at or near the heel of the dam. Uplift on flat and inclined bedding planes will be dependent on their state of compression and the presence of drains passing through these planes as described for dams in Chapter 3. Passive resistance will be based on the rock conditions downstream of the dam. Adversely inclined joints, faults, rock fracturing, or damage from excavation by blasting will affect available passive resistance.

b. Anchor penetration. Required anchor penetration depends on the purpose of the anchor. Anchors provided to resist uplift of the heel must have sufficient penetration to develop the capacity of the anchors. Anchors provided to resist sliding must be fully developed below the lowest critical sliding plane. Critical sliding planes are those requiring anchors to meet minimum acceptable factors of safety against sliding.

c. Anchor resistance. The capacity of the anchor to resist uplift should be limited to the force that can be developed by the submerged weight of the rock engaged by the anchor. Rock engaged will either be shaped as cones or intersecting cones depending on the length and spacing of the anchors. The anchor force that can be developed should be based on the pullout resistance of a cone with an apex angle of 90 deg. Tensile stresses will occur in the anchorage zone of prestressed anchors. The possibility of foundation cracking as a result of these tensile stresses must be considered. It is possible that cracks in the foundation could open at the lower terminal points of the anchors and propagate downstream. To alleviate this potential problem, a sufficient weight of submerged rock should be engaged to resist the anchor force, and the anchor depths should be staggered.

8-8. Example Problem

The following example is a gated outlet structure for an earth fill dam. The existing gated spillway monoliths are deficient in sliding resistance along a weak seam in the foundation which daylights in the stilling basin. A cross-section of the spillway monoliths is shown in Figure 8-1. The spillway monoliths are founded at elevation 840 on moderately hard silty shale. A continuous soft, plastic clay shale seam approximately 1/2 inch in thickness exists at elevation 830. A free body diagram showing forces acting on the gravity structure and foundation above the weak seam is shown in Figure 8-2. Even though the foundation drains penetrate the potential sliding plane, the drains are assumed ineffective as they are insufficient to drain a thin clay seam. The sliding plane is in full compression, and uplift is assumed to vary uniformly from upper pool head to zero in the stilling basin. A drained shear strength of $20° 30'$ has been assigned to this potential sliding surface, and a sliding factor of safety of 0.49 has been calculated for loading condition No. 2, i.e., pool to top of closed spillway gates. The tailwater is below the level of the sliding surface. A summary of loads and the resulting factor of safety for this critical loading condition is shown in Table 8-1. The design of anchors to provide a required factor of safety of 1.70 is summarized in Table 8-2. The anchors are located as shown in Figure 8-3. Details of the anchors are shown in Figure 8-4. The 45-deg angle for the anchors was selected to minimize drilling and to provide a large component of resisting force without creating a potential upstream sliding problem during low pools. Tips of anchors are staggered to avoid tensile stress concentrations in the foundation. The anchors are embedded below the lowest sliding plane requiring anchors to meet required safety factors. Reinforcement similar to that used in post-tensioned beams is provided under the bearing plates to resist the high tensile bursting stresses associated with large capacity anchors. The anchors were tensioned in the sequence shown in Figure 8-4 to avoid unacceptable stress concentrations in the concrete monoliths. The anchors were designed, installed, and tested in accordance with PTI (1985). The anchors are designed for a working load of 826 kips and were locked-off at 910 kips (i.e., working load plus 10 percent) to allow for calculated relaxation of the anchors, creep in the concrete structure, and consolidation of the foundation. Proof testing of all anchors to 80 percent of ultimate strength confirmed the adequacy of the anchors for a working load of 826 kips per anchor (approximately 60 percent of ultimate strength). Each anchor successfully passed a 14th day lift-off test, secondary grouting was accomplished, and anchor head recesses were filled with concrete to restore the spillway profile.
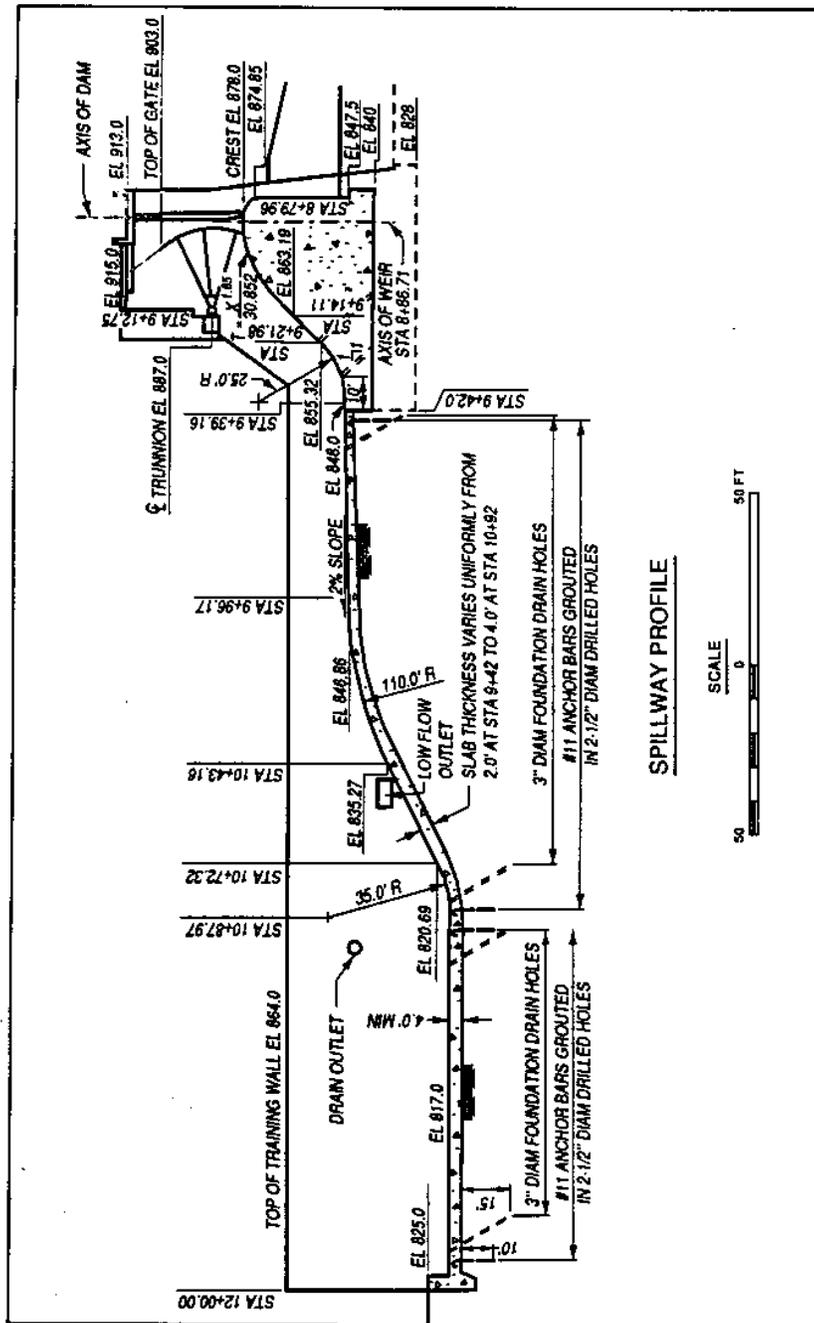
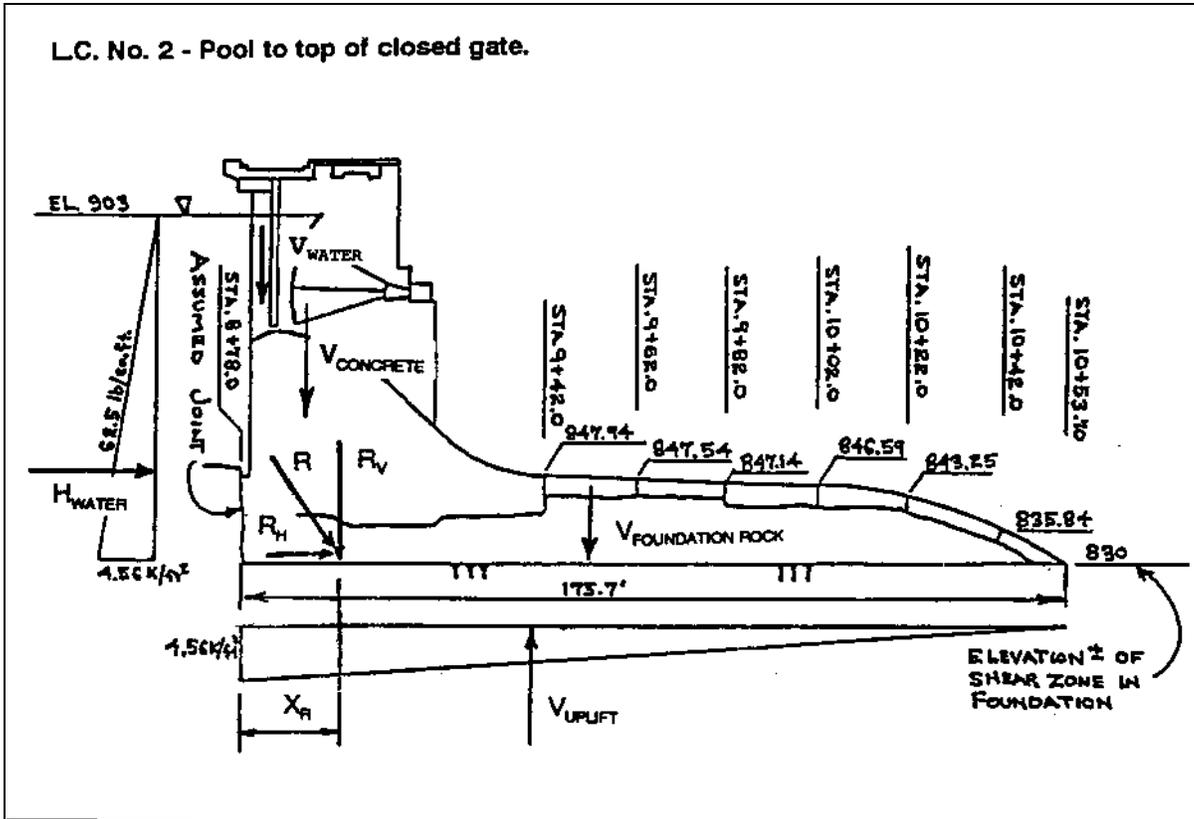Figure 8-1. Cross-section of spillway monoliths

Figure 8-2. Free body diagram, R_y = resultant of vertical forces, R_H = resultant of horizontal forces, and X_R = distance from heel to resultant location on sliding plane

Table 8-1
Summary of Forces on the Sliding Plane. Loading Condition No. 2 (Pool at Top of Gates, Tailwater Below Sliding Surface)

	Σ Vert, kips	Σ Horz, kips
Concrete	11,910	
Rock (Saturated Weight)	13,160	
Machinery	10	
Gates	70	
Water Down	870	
Water Up	- 90	
Uplift	- 16,830	
Horizontal Water		6,990
	_____	_____
Totals, Loading Condition No. 2	9,100	6,990

$$\textit{Sliding FS, Without Anchors} = \frac{\textit{TAN } 20.5° \times 9,100}{6,990} = 0.49$$

Table 8-2
Summary of Forces on the Sliding Plane. Loading Condition No. 2, With Anchors

	Σ Vert, kips	Σ Horz, kips
Concrete	11,910	
Rock	13,160	
Machinery	10	
Gates	70	
Water Down	870	
Water Up	- 90	
Uplift	- 16,830	
Horizontal Water		6,990
Anchors (Vertical) 7 x 826 x 0.707	4,088	
Anchors (Horizontal)		- 4,088
	_____	_____
Totals, Loading Condition No. 2	13,188	2,902

$$\textit{Sliding FS, With Anchors} = \frac{TAN\ 20.5° \times 13,188}{6,990 - 4,088} = 1.70$$
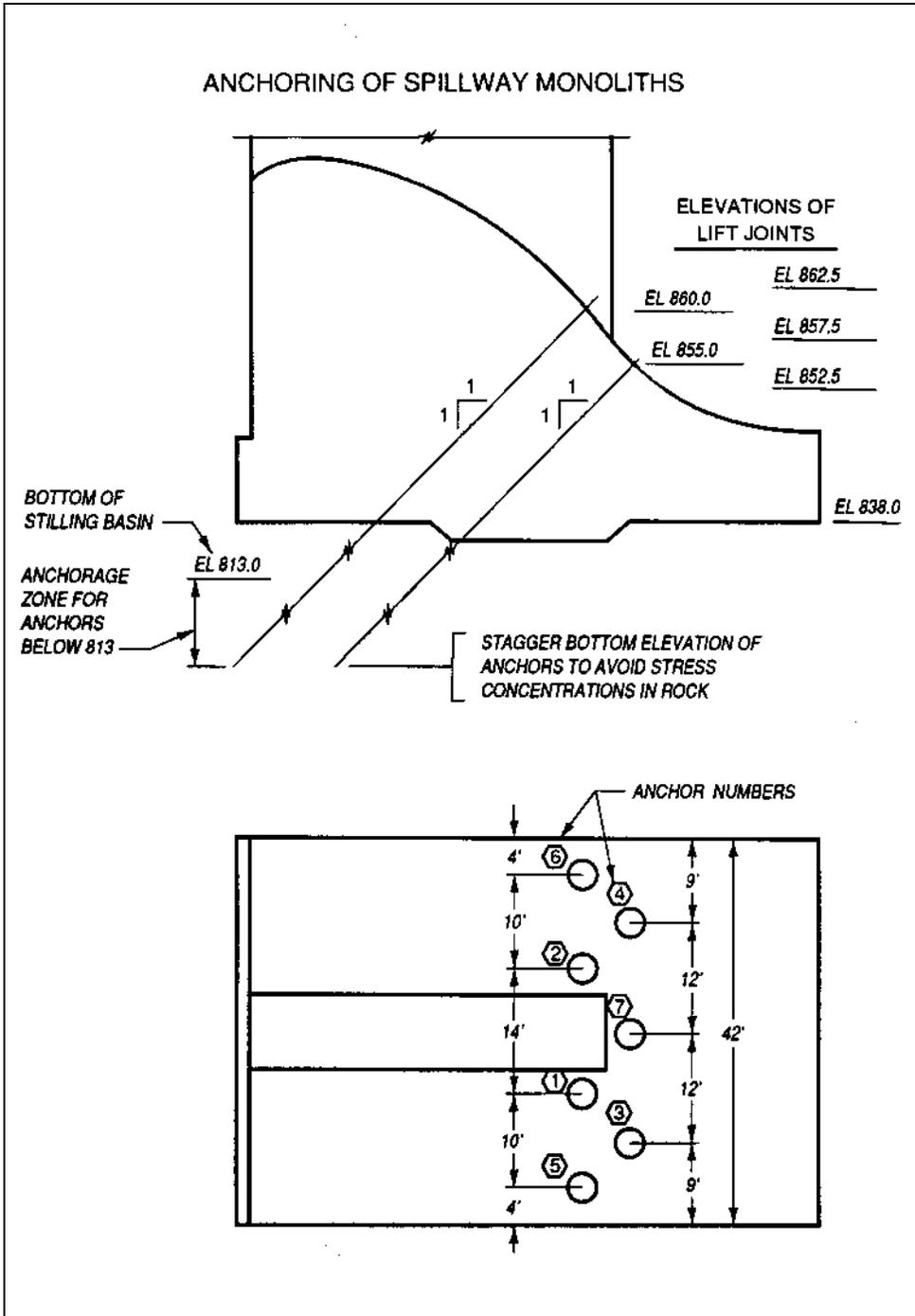
Figure 8-3.   Location of anchors
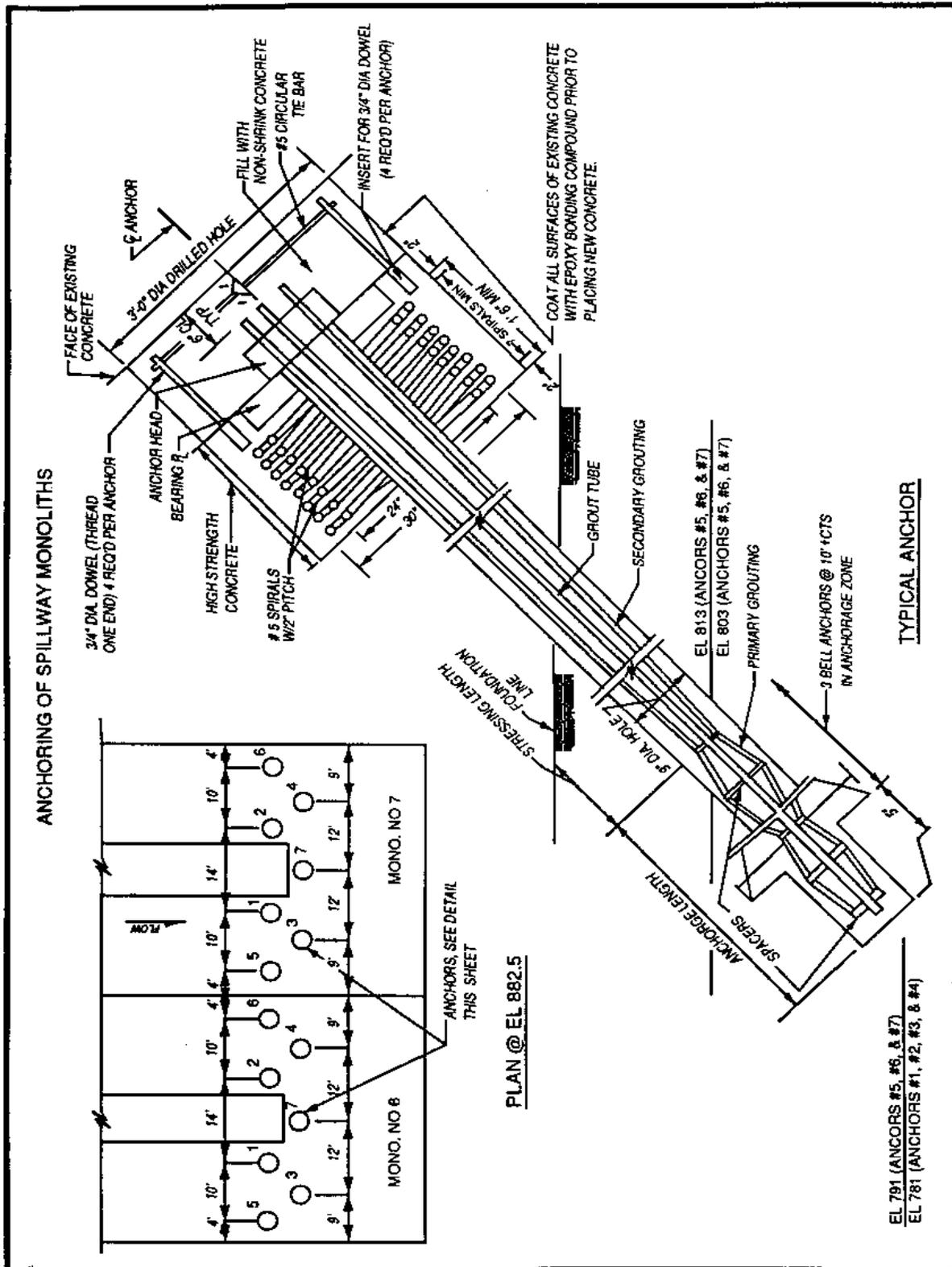
Figure 8-4. Details of anchors

Chapter 9
Roller-Compacted Concrete Gravity Dams

9-1. Introduction

Gravity dams built using the RCC construction method, afford economies over conventional concrete through rapid placement techniques. Construction procedures associated with RCC require particular attention be given in the layout and design to watertightness and seepage control, horizontal and transverse joints, facing elements, and appurtenant structures. The designer should take advantage of the latitude afforded by RCC construction and use engineering judgment to balance cost reductions and technical requirements related to safety, durability, and long-term performance. A typical cross section of an RCC dam is shown in Figure 9-1. RCC mix design and construction should be in accordance with EM 1110-2-2006.
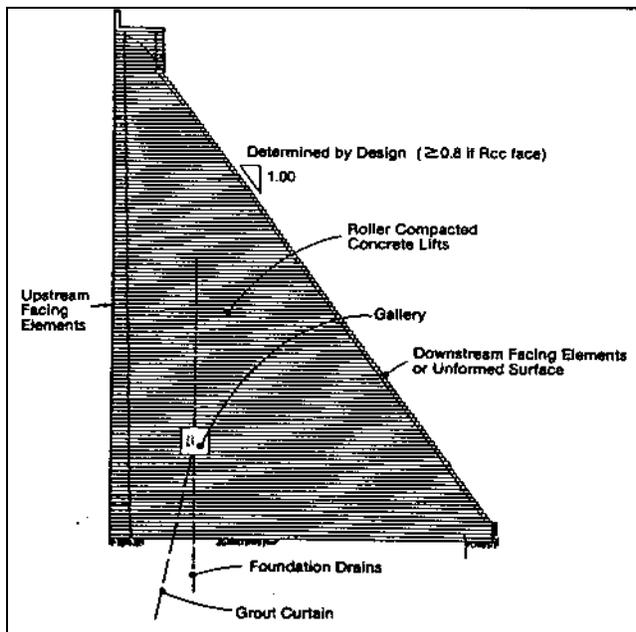

Figure 9-1. Typical RCC dam section

9-2. Construction Method

Construction techniques used for RCC placement often result in a much lower unit cost per cubic yard compared with conventional concrete placement methods. The dry, nonflowable nature of RCC makes the use of a wide range of equipment for construction and continuous placement possible. End and bottom dump trucks and/or conveyors can be used for transporting concrete from the mixer to the dam. Mechanical spreaders, such as caterpillars and graders, place the material in layers or lifts. Self-propelled, vibratory, steel-wheeled, or pneumatic rollers along with the dozers perform the compaction. The thickness of the placement layers, ranging from 8 to 24 inches, is established by the compaction capabilities. With the flexibility of using the above equipment and continuous placement, RCC dams can be constructed at significantly higher rates than those achievable with conventional mass concrete. A typical work layout for the RCC placement spreading operation is illustrated in Figure 9-2.

9-3. Economic Benefits

RCC construction techniques have made gravity dams an economically competitive alternative to embankment structures. The following factors tend to make RCC more economical than other dam types:

a. Material savings. Construction cost histories of RCC and conventional concrete dams show the unit cost per cubic yard of RCC is considerably less. The unit cost of concrete for both types of dam varies with the volume of the material in the dam. As the volume increases, the unit cost decreases. The cost savings of RCC increase as the volume decreases. RCC dams have considerably less volume of construction material than embankments of the same height. As the height increases, the volume versus height for the embankment dam increases almost exponentially in comparison to the RCC dam. Thus, the higher the structure, the more likely the RCC dam will be less costly than the embankment alternative.

b. Rapid construction. The rapid construction techniques and reduced concrete volume account for the major cost savings in RCC dams. Maximum placement rates of 5,800 to 12,400 cubic yards/day have been achieved. These production rates make dam construction in one construction season readily achievable. When compared with embankment dams, construction time is reduced by 1 to 2 years. Other benefits from rapid construction include reduced construction administration costs, earlier project benefits, and possible selection of sites with limited construction seasons. Basically, RCC construction offers economic advantages in all aspects of dam construction that are related to time.

c. Spillways and appurtenant structures. The location and layout alternatives for spillways, outlet and hydropower works, and other appurtenant structures in
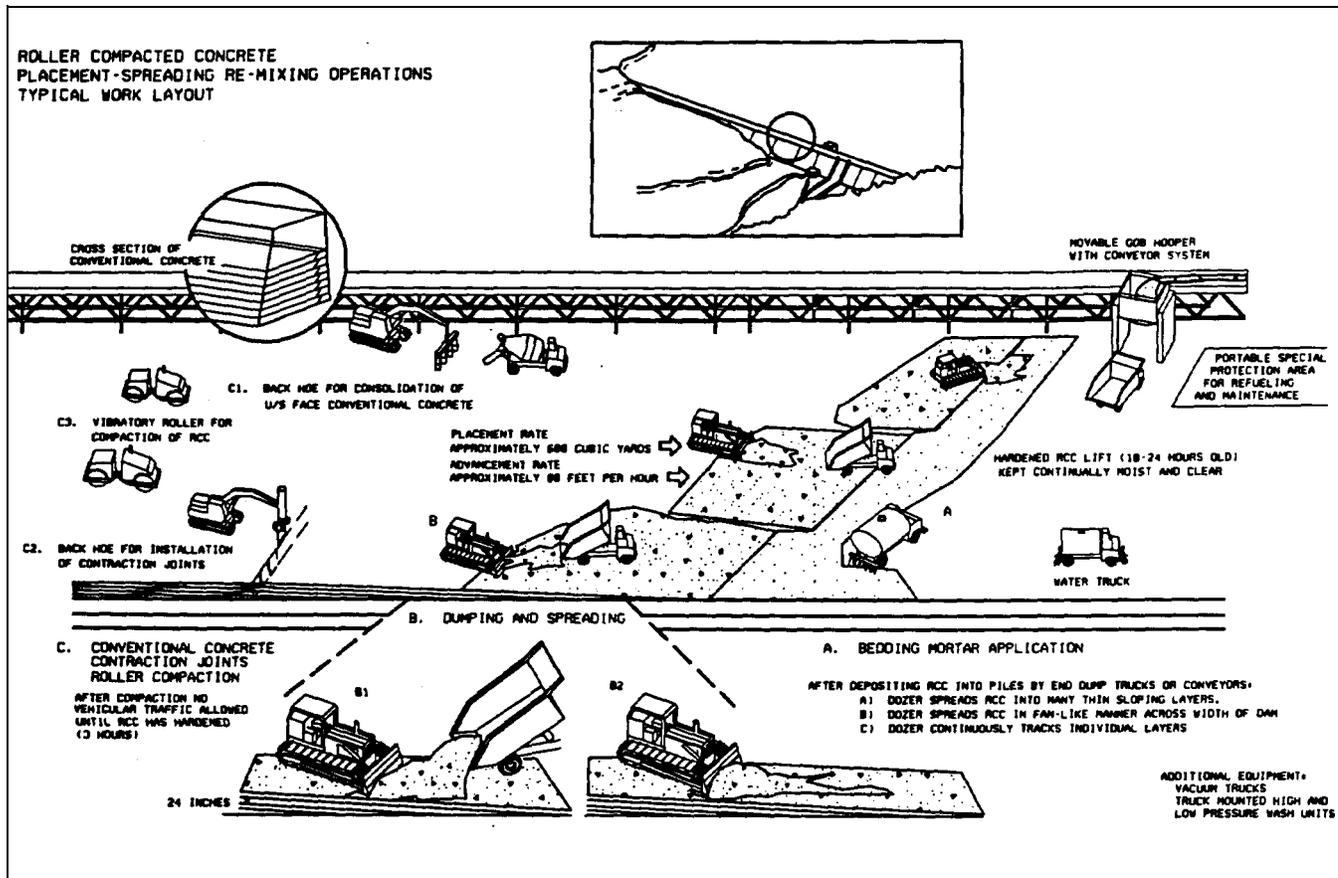
Figure 9-2. Typical work layout for RCC placement spreading operation

RCC dams provide additional economic advantages compared with embankment dams. The arrangements of these structures is similar to conventional concrete dams, but with certain modifications to minimize costly interference to the continuous RCC placement operation. Gate structures and intakes should be located outside the dam mass. Galleries, adits, and other internal openings should be minimized. Details on the layout and design of spillways and appurtenant structures are discussed in paragraph 9-4. Spillways for RCC dams can be directly incorporated into the structure. The layout allows discharging flows over the dam crest and down the downstream face. In contrast, the spillway for an embankment dam is normally constructed in an abutment at one end of the dam or in a nearby natural saddle. Generally, the embankment dam spillway is more costly. For projects that require a multiple-level intake for water quality control or for reservoir sedimentation, the intake structure can be readily anchored to the upstream face of the dam. For an embankment dam, the same type of intake tower is a freestanding tower in the reservoir or a structure built into or on the reservoir side of the abutment. The economic

savings for an RCC dam intake is considerably cheaper, especially in high seismic areas. The shorter base dimension of an RCC dam compared with an embankment dam reduces the size and length of the conduit and penstock for outlet and hydropower works.

d. Diversion and cofferdam. RCC dams provide cost advantages in river diversion during construction and reduce damages and risks associated with cofferdam overtopping. The diversion conduit will be shorter compared with embankment dams. With a shorter construction period, the size of the diversion conduit and cofferdam height can be reduced. These structures may need to be designed only for a seasonal peak flow instead of annual peak flows. With the high erosion resistance of RCC, if overtopping of the cofferdam did occur, the potential for a major failure would be minimal and the resulting damage would be less.

e. Other advantages. The smaller volume of an RCC dam makes the construction material source less of a driving factor in site selection of a dam. Furthermore, the

borrow source will be considerably smaller and more environmentally acceptable. The RCC dam is also inherently safer against internal erosion, overtopping, and seismic ground motions.

9-4. Design and Construction Considerations

a. Watertightness and seepage control. Achieving watertightness and controlling seepage through RCC dams are particularly important design and construction considerations. Excessive seepage is undesirable from the aspect of structural stability and because of the adverse appearance of water seeping on the downstream dam face, the economic value associated with lost water, and possible long-term adverse impacts on durability. RCC that has been properly proportioned, mixed, placed, and compacted should be as impermeable as conventional concrete. The joints between the concrete lifts and interface with structural elements are the major pathways for potential seepage through the RCC dam. This condition is primarily due to segregation at the lift boundaries and discontinuity between successive lifts. It can also be the result of surface contamination and excessive time intervals between lift placements. Seepage can be controlled by incorporating special design and construction procedures that include contraction joints with waterstops making the upstream face watertight, sealing the interface between RCC layers, and draining and collecting the seepage.

b. Upstream facing. RCC cannot be compacted effectively against upstream forms without the forming of surface voids. An upstream facing is required to produce a surface with good appearance and durability. Many facings incorporate a watertight barrier. Facings with barriers include the following:

(1) Conventional form work with a zone of conventional concrete placed between the forms and RCC material.

(2) Slip-formed interlocking conventional concrete elements. RCC material is compacted against the cured elements.

(3) Precast concrete tieback panels with a flexible waterproof membrane placed between the RCC and the panels.

A waterproof membrane sprayed or painted onto a conventional concrete face is another method; however, its use has been limited since such membranes are not elastic enough to span cracks that develop and because of concerns about moisture developing between the membrane and face and subsequent damage by freezing.

c. Horizontal joint treatment. Bond strength and permeability are major concerns at the horizontal lift joints in RCC. Good sealing and bonding are accomplished by improving the compactibility of the RCC mixture, cleaning the joint surface, and placing a bedding mortar (a mixture of cement paste and fine aggregate) between lifts. When the placement rate and setting time of RCC are such that the lower lift is sufficiently plastic to blend and bond with the upper layer, the bedding mortar is unnecessary; however, this is rarely feasible in normal RCC construction. Compactibility is improved by increasing the amount of mortar and fines in the RCC mixture. The lift surfaces should be properly moist cured and protected. Cleanup of the lift surfaces prior to RCC placement is not required as long as the surfaces are kept clean and free of excessive water. Addition of the bedding mortar serves to fill any voids or depressions left in the surface of the previous lift and squeezes up into the voids in the bottom of the new RCC lift as it is compacted. A bedding mix consisting of a mixture of cement paste and fine and 3/8-in.-MSA aggregate is also applied at RCC contacts with the foundation, abutment surfaces, and any other hardened concrete surfaces. EM 1110-2-2006 contains additional guidance on this issue.

d. Seepage collection. A collection and drainage system is a method for stopping unsightly seepage water from reaching the downstream face and for preventing excessive hydrostatic pressures against conventional concrete spillway or downstream facing. It will also reduce uplift pressures within the dam and increase stability. Collection methods include vertical drains with waterstops at the upstream face and vertical drain holes drilled from within the gallery near the upstream or downstream face. Collected water can be channeled to a gallery or the dam toe.

e. Nonoverflow downstream facing. Downstream facing systems for nonoverflow sections may be required for aesthetic reasons, maintaining slopes steeper than the natural repose of RCC, and freeze-thaw protection in severe climate locations. Facing is necessary when the slope is steeper than 0.8H to 1.0V when lift thickness is limited to 12 inches or less. Thicker lifts require a flatter slope. Experience has demonstrated that these are the steepest uncompacted slopes that can be practically controlled without special equipment or forms. The exposed edge of an uncompacted slope will have a rough stair-stepped natural gravel appearance with limited strength within 12 inches of the face. Downstream facing systems

include conventional vertical slipforming placement and horizontal slipforming similar to that used on the upstream face. When this type of slope is used, the structural cross section should include a slight overbuild to account for deterioration and unraveling of material loosened from severe weather exposure over the project life (see Figure 9-3). Several recent projects have compacted downstream faces using a tractor-mounted vibrating plate.
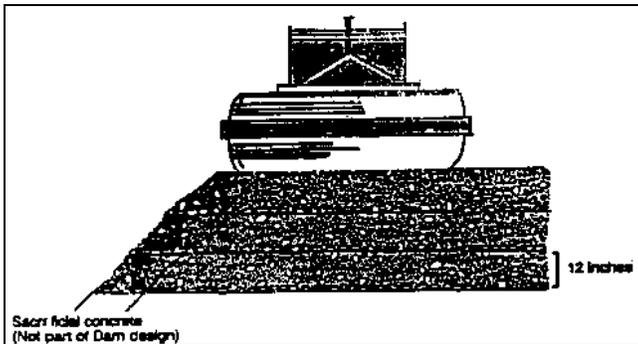

Figure 9-3. Compaction of RCC at downstream face

f. Transverse contraction joints. Transverse contraction joints are required in most RCC dams. The potential for cracking may be slightly lower in RCC because of the reduction in mixing water and reduced temperature rise resulting from the rapid placement rate and lower lift heights. In addition, the RCC characteristic of point-to-point aggregate contact decreases the volume shrinkage. Thermal cracking may, however, create a leakage path to the downstream face that is aesthetically undesirable. Thermal studies should be performed to assess the need for contraction joints. Contraction joints may also be required to control cracking if the site configuration and foundation conditions may potentially restrain the dam. If properly designed and installed, contraction joints will not interfere or complicate the continuous placement operation of RCC. At Elk Creek Dam, contraction joints were installed with no impact to RCC placement operations by inserting galvanized steel sheeting into the uncompacted RCC for the entire thickness and height of the dam. The sheets were pushed vertically into the RCC by means of a tractor-mounted vibratory blade, as shown in Figure 9-4.
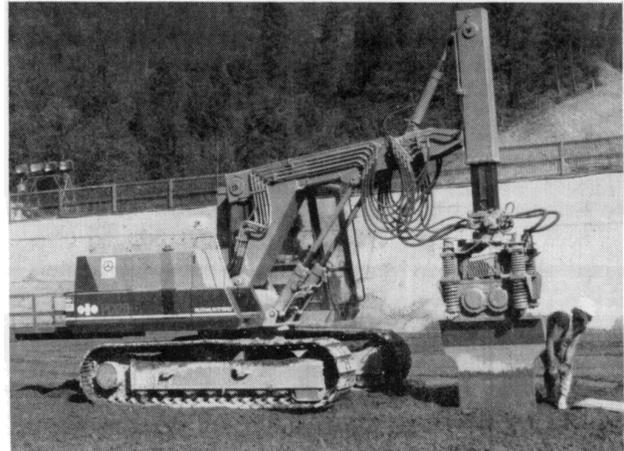

Figure 9-4. Contract joint placement using a vibrating blade to insert galvanized steel sheeting

g. Waterstops. Standard waterstops may be installed in an internal zone of conventional concrete placed around the joint near the upstream face. Waterstops and joint drains are installed in the same manner as for conventional concrete dams. Typical internal waterstops and joint drain construction in RCC dams are shown in Figure 9-5. Around galleries and other openings crossing joints, waterstop installation will require a section of conventional concrete around the joint.
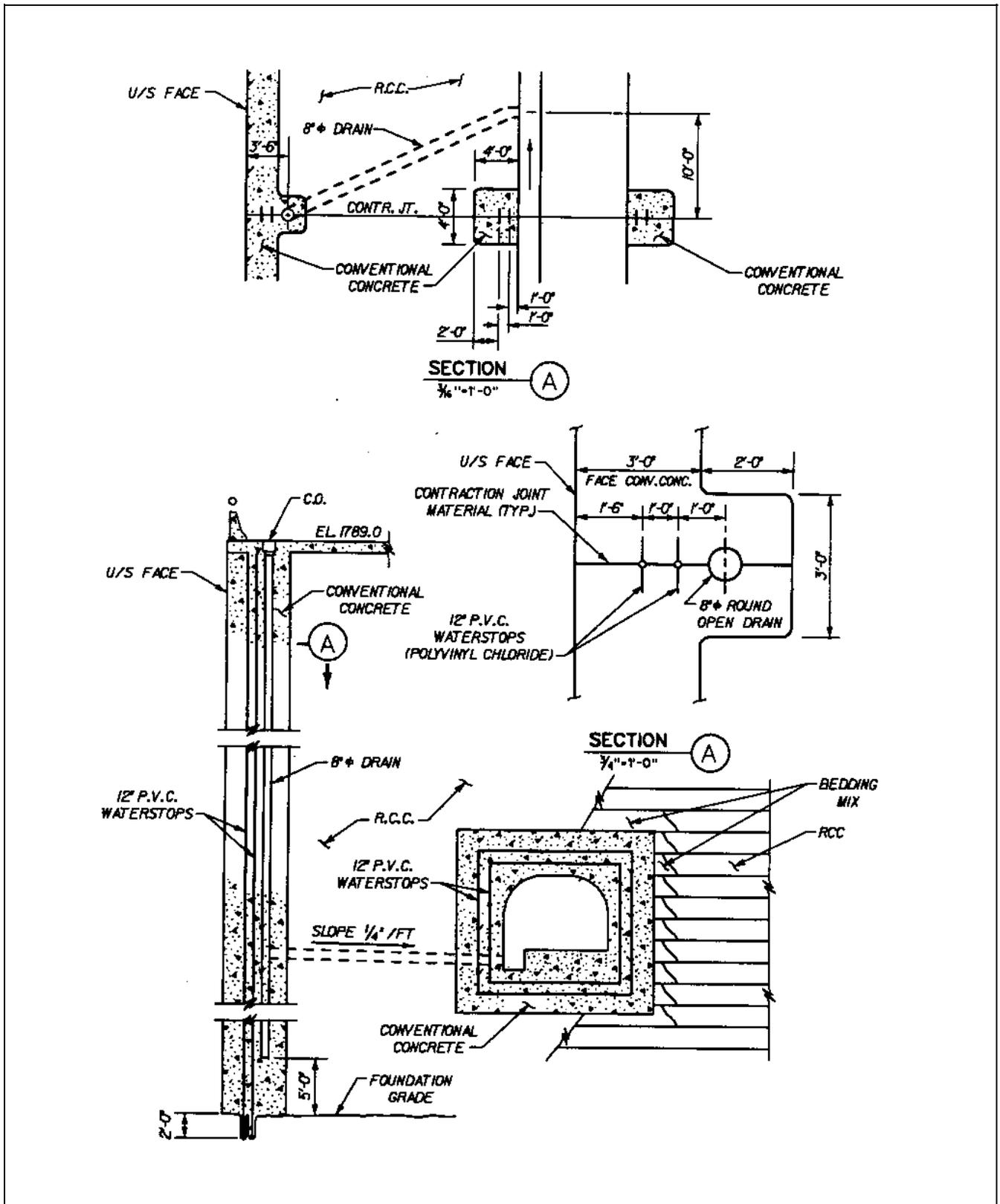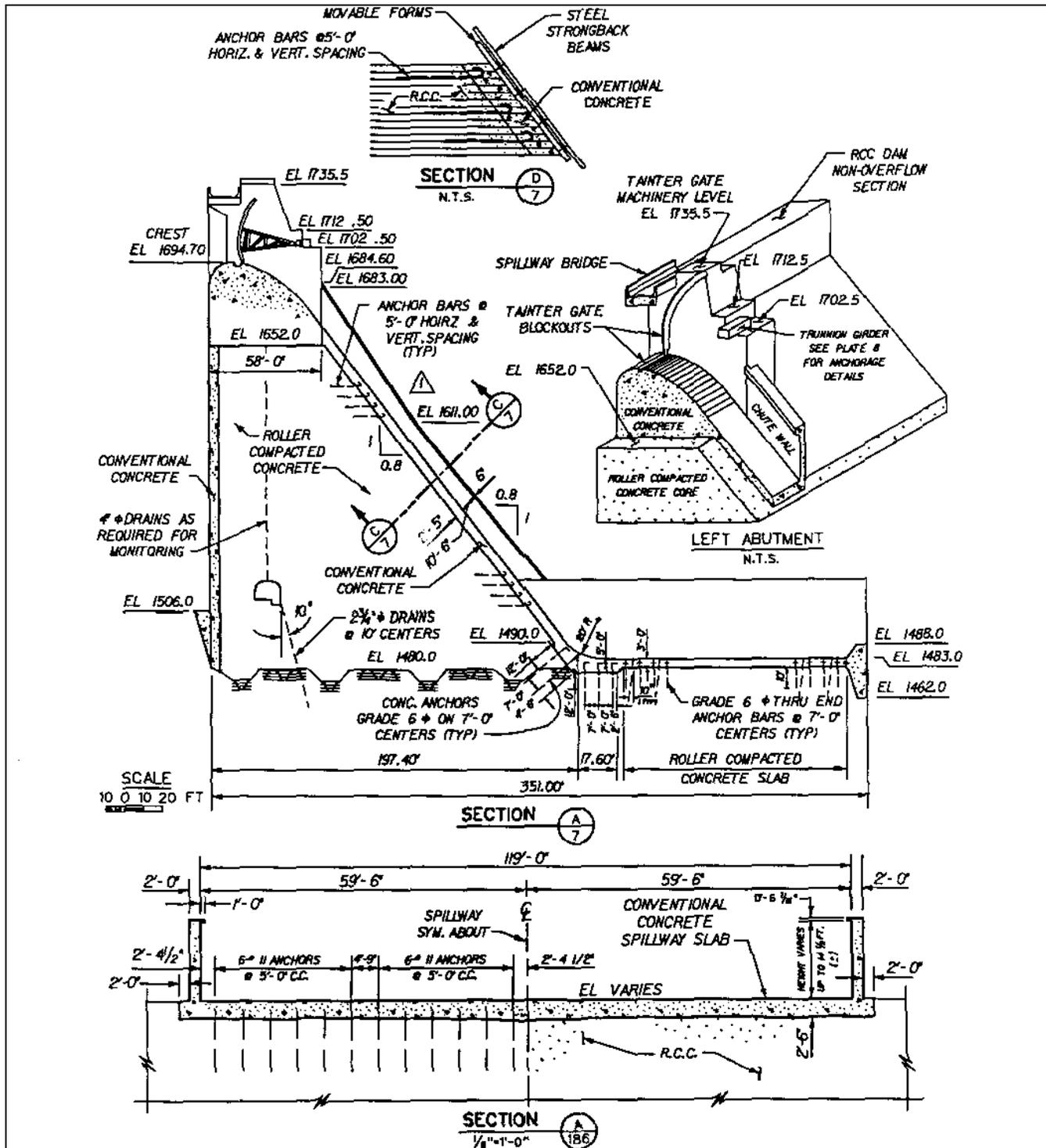
Figure 9-5. Typical internal waterstops and joint drain construction in RCC dams

Figure 9-6. RCC spillway details