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ARCH DAM DESIGN</td><td></td></tr>
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Manual
No. 1110-2-2201 31 May 1994

Engineering and Design
ARCH DAM DESIGN

1. Purpose. This manual provides information and guidance on the design, analysis, and construction of concrete arch dams.

2. Applicability. This manual applies to HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities (FOA) having civil works responsibilities.

3. Discussion. This manual provides general information, design criteria and procedures, static and dynamic analysis procedures, temperature studies, concrete testing requirements, foundation investigation requirements, and instrumentation and construction information for the design of concrete arch dams.

FOR THE COMMANDER:

WILLIAM D. BROWN
Colonel, Corps of Engineers
Chief of Staff

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
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CHAPTER 1

INTRODUCTION

1-1. Purpose and Scope.

a. This manual provides general information, design criteria and procedures, static and dynamic analysis procedures, temperature studies, concrete testing requirements, foundation investigation requirements, and instrumentation and construction information for the design of concrete arch dams. The guidance provided in this manual is based on state of the art in this field as practiced at the time of publication. This manual will be updated as changes in design and analysis of arch dams are developed. The information on design and analysis presented in this manual is only applicable to arch dams whose horizontal and vertical sections are bounded by one or more circular arcs or a combination of straight lines and circular arcs.

b. This manual is a product of the Arch Dam Task Group which is a component of the Computer-Aided Structural Engineering (CASE) Program of the U.S. Army Corps of Engineers (USACE). Task group members are from the USACE, U.S. Bureau of Reclamation (USBR), and the Federal Energy Regulatory Commission (FERC). Individual members and others contributing to this manual are as follows: Donald R. Dressler (CECW-ED), Jerry L. Foster (CECW-ED), G. Ray Navidi (CEORH-ED), Terry W. West (FERC), William K. Wigner (CESAJ-EN), H. Wayne Jones (CEWES-IM), Byron J. Foster (CESAD-EN), David A. Dollar (USBR), Larry K. Nuss (USBR), Howard L. Boggs (USBR, retired/consultant), Dr. Yusof Ghanaat (QUEST Structures/consultant) and Dr. James W. Erwin (USACE, retired/consultant).

c. Credit is given to Mr. Merlin D. Copen (USBR, retired) who inspired much of the work contained in this manual. Mr. Copen's work as a consultant to the U.S. Army Engineer District, Jacksonville, on the Corps' first double-curved arch dam design, Portugues Arch Dam, gave birth to this manual. Professor Ray W. Clough, Sc. D. (Structures consultant), also a consultant to the Jacksonville District for the design of the Portugues Arch Dam, provided invaluable comments and recommendations in his review and editing of this manual.

1-2. Applicability.

This manual is applicable to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having civil works responsibilities.

1-3. References and Related Material.

a. References. References are listed in Appendix A.

b. Related Material. In conjunction with this manual and as part of the Civil Works Guidance Update Program, a number of design and analysis tools have been developed or enhanced for use by USACE districts. A brief description is as follows:

(1) Arch Dam Stress Analysis System (ADSAS) (U.S. Bureau of Reclamation (USBR) 1975). This is the computerized version of the trial load method of analyzing arch dams developed by the Bureau of Reclamation. ADSAS has been converted from mainframe to PC and a revised, user-friendly manual has been prepared. ADSAS is a powerful design tool which has been used in the design of most modern arch dams in the United States.

(2) Graphics-Based Dam Analysis Program (GDAP) (Ghanaat 1993a). GDAP is a finite element program for static and dynamic analysis of concrete arch dams based on the Arch Dam Analysis Program (ADAP) that was developed by the University of California for the USBR in 1974. The GDAP program is PC-based and has graphics pre- and postprocessing capabilities. The finite element meshes of the dam, foundation rock, and the reservoir are generated automatically from a limited amount of data. Other general-purpose finite element method (FEM) programs can also be used for the analysis of arch dams.

(3) Interactive Graphics Layout of Arch Dams (IGLAD) is an interactive PC-based program for the layout of double-curvature arch dams. The program enables the designer to prepare a layout, perform necessary adjustments, perform stress analyses using ADSAS, and generate postprocessing graphics and data. This program was developed by the USACE.

1-4. Definitions. Terminology used in the design and analysis of arch dams is not universal in meaning. To avoid ambiguity, descriptions are defined and shown pictorially, and these definitions will be used throughout this manual.

a. Arch (Arch Unit). Arch (or arch unit) refers to a portion of the dam bounded by two horizontal planes, 1 foot apart. Arches may have uniform thickness or may be designed so that their thickness increases gradually on both sides of the reference plane (variable thickness arches).

b. Cantilever (Cantilever Unit). Cantilever (or cantilever unit) is a portion of the dam contained between two vertical radial planes, 1 foot apart.

c. Extrados and Intrados. The terminology most commonly used in referring to the upstream and downstream faces of an arch dam is extrados and intrados. Extrados is the upstream face of arches and intrados is the downstream face of the arches. These terms are used only for the horizontal (arch) units; the faces of the cantilever units are referred to as upstream and downstream, as appropriate. See Figure 1-1 for these definitions.

d. Site Shape. The overall shape of the site is classified as a narrow-V, wide-V, narrow-U, or wide-U as shown in Figure 1-2. These terms, while being subjective, present the designer a visualization of a site form from which to conceptually formulate the design. The terms also help the designer to develop knowledge and/or experience with dams at other sites. Common to all arch dam sites is the crest length-to-height ratio, cl:h. Assuming for comparison that factors such as central angle and height of dam are equal, the arches of dams designed for wider canyons would be more flexible in relation to cantilever stiffness than those of dams in narrow canyons, and a proportionately larger part of the load would be carried by cantilever action.

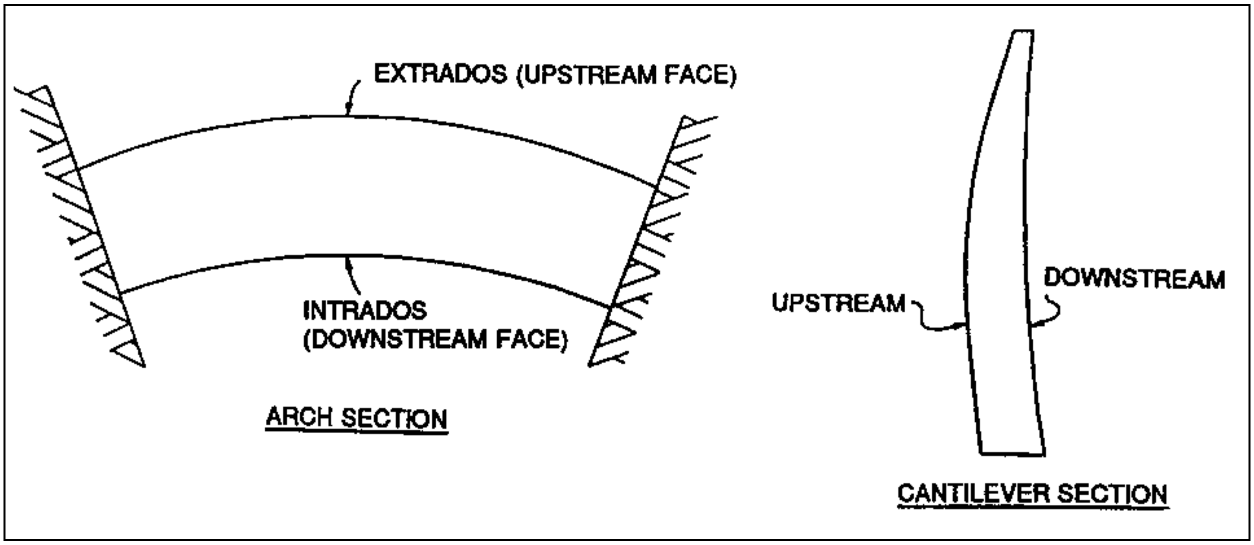

Figure 1-1. Typical arch unit and cantilever unit

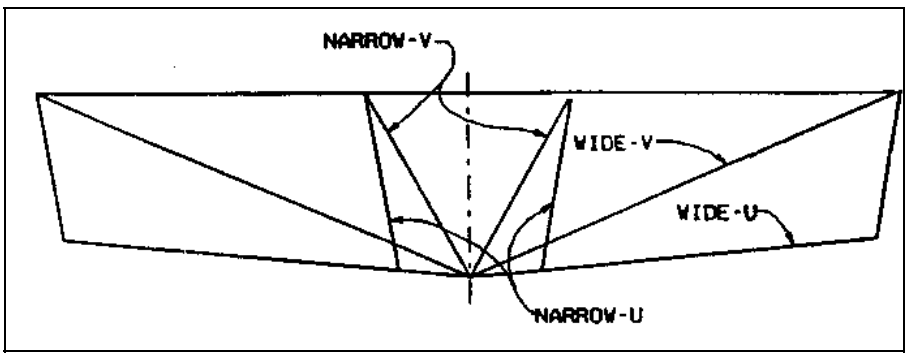

Figure 1-2. Schematic profiles of various dam sites

e. Crest Length-Height Ratio. The crest length-to-height ratios of dams may be used as a basis for comparison of proposed designs with existing conditions and with the relative effects of other controlling factors such as central angle, shape of profile, and type of layout. The length-to-height ratio also gives a rough indication of the economic limit of an arch dam as compared with a dam of gravity design (Figure 1-2). See paragraph 2-1b for general guidelines.

f. Narrow-V. A narrow-V site would have a cl:h of 2:1 or less. Such canyon walls are generally straight, with few undulations, and converge to a narrow streambed. This type of site is preferable for arch dams since the applied load will be transferred to the rock predominantly by arch action. Arches will be generally uniform in thickness, and the cantilevers will be nearly vertical with some slight curvature at the arch crown. Faces most likely will be circular in plan, and the dam will be relatively thin. From the standpoint of avoiding excessive tensile stresses in the arch, a type of layout should be used which will provide as much curvature as possible in the arches. In some sites, it may be necessary to use variable-thickness arches with a variation in location of circular arc centers to produce greater curvature in the lower arches. Figure 1-4 shows an example of a two-centered variable-thickness arch dam for a nonsymmetrical site.

g. Wide-V. A wide-V site would have a cl:h of 5:1 or more. The upper limit for cl:h for arch dams is about 10:1. Canyon walls will have more pronounced undulations but will be generally straight after excavation, converging to a less pronounced v-notch below the streambed. Most of the live load will be transferred to rock by arch action. Arches will generally be uniform in thickness with some possible increase in thickness near the abutments. The "crown" (central) cantilever will have more curvature and base thickness than that in a narrow-V of the same height. In plan, the crest most likely would be three-centered and would transition to single-center circular arches at the streambed. Arches would be thicker than those in the narrow-V site.

h. Narrow-U. In narrow-U sites, the canyon walls are near vertical in the upper half of the canyon. The streambed width is fairly large, i.e., perhaps one-half the canyon width at the crest. Above 0.25h, most of the live load will be transferred to rock by arch action. Below 0.25h, the live load will increasingly be supported by cantilever action toward the lowest point. There the cantilevers have become stubby while the arches are still relatively long. The upper arches will be uniform in thickness but become variable in thickness near the streambed. The crown cantilever will have more curvature than the crown cantilever in a narrow-V site of equal height. Faces will generally be circular in plan. Arches will be thin because of the narrow site. In dams constructed in U-shaped canyons, the lower arches have chord lengths almost as long as those near the top. In such cases, use of a variable-thickness arch layout will normally give a relatively uniform stress distribution. Undercutting on the upstream face may be desirable to eliminate areas of tensile stress at the bases of cantilevers.

i. Wide-U. Wide-U sites are the most difficult for an arch dam design because most of the arches are long compared to the crest length. In the lower 0.25h, much of the live load is carried by cantilever action because the long flexible arches carry relatively little load. In this area, cantilever thickness tends to increase rapidly to support the increased water pressure. Arch thickness variation in the horizontal direction may range from uniform at the crest to variable at the streambed. The transition will most likely occur at about the upper one-third level. The crown cantilever here should have the most curvature of any type of site.

j. Reference Plane. As shown in Figures 1-3 through 1-5, the reference plane is a vertical radial plane usually based in the streambed. The reference plane contains the crown cantilever and the loci of the central centers as shown in Figure 1-6. It is from this plane that the angle to the arch abutment is measured. Also shown are the axis and axis center. The axis is a vertical surface curved in plan intersecting the crown cantilever at the crest and upstream face. The axis is developed in plan by the axis radius which is the distance between the axis and the axis center located downstream. A method of estimating values for these terms will be described in a later section. The reference plane will theoretically consist of one, two, or three planes of centers. One plane of centers is used to describe arches in a symmetrical site as shown in Figure 1-3. Two planes of centers are used to describe arches in nonsymmetrical sites as shown in Figure 1-4. Three planes

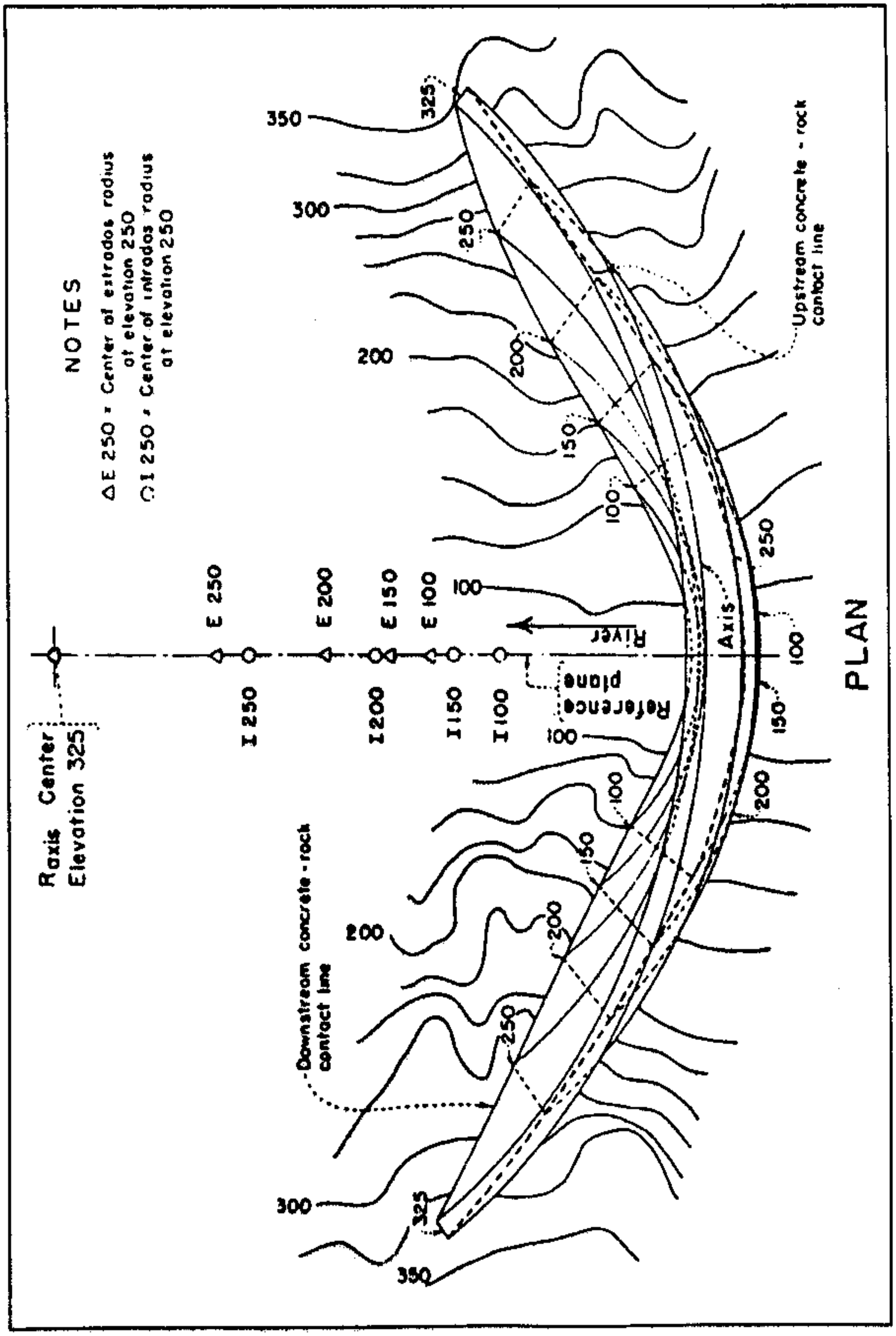

Figure 1-3. Typical single-center variable thickness arch dam in a symmetrical site

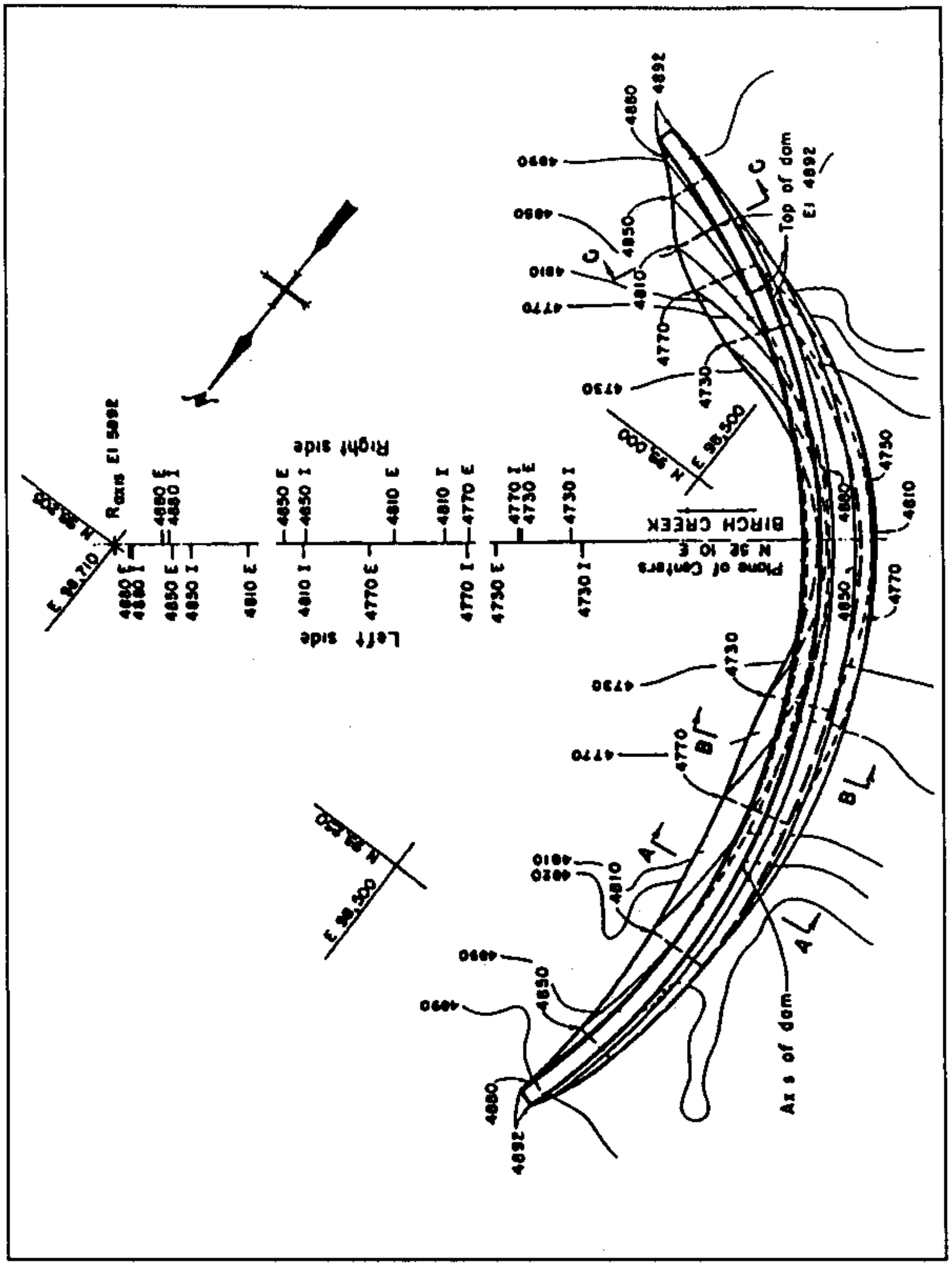

Figure 1-4. Typical two-centered variable-thickness arch dam in a nonsymmetrical site

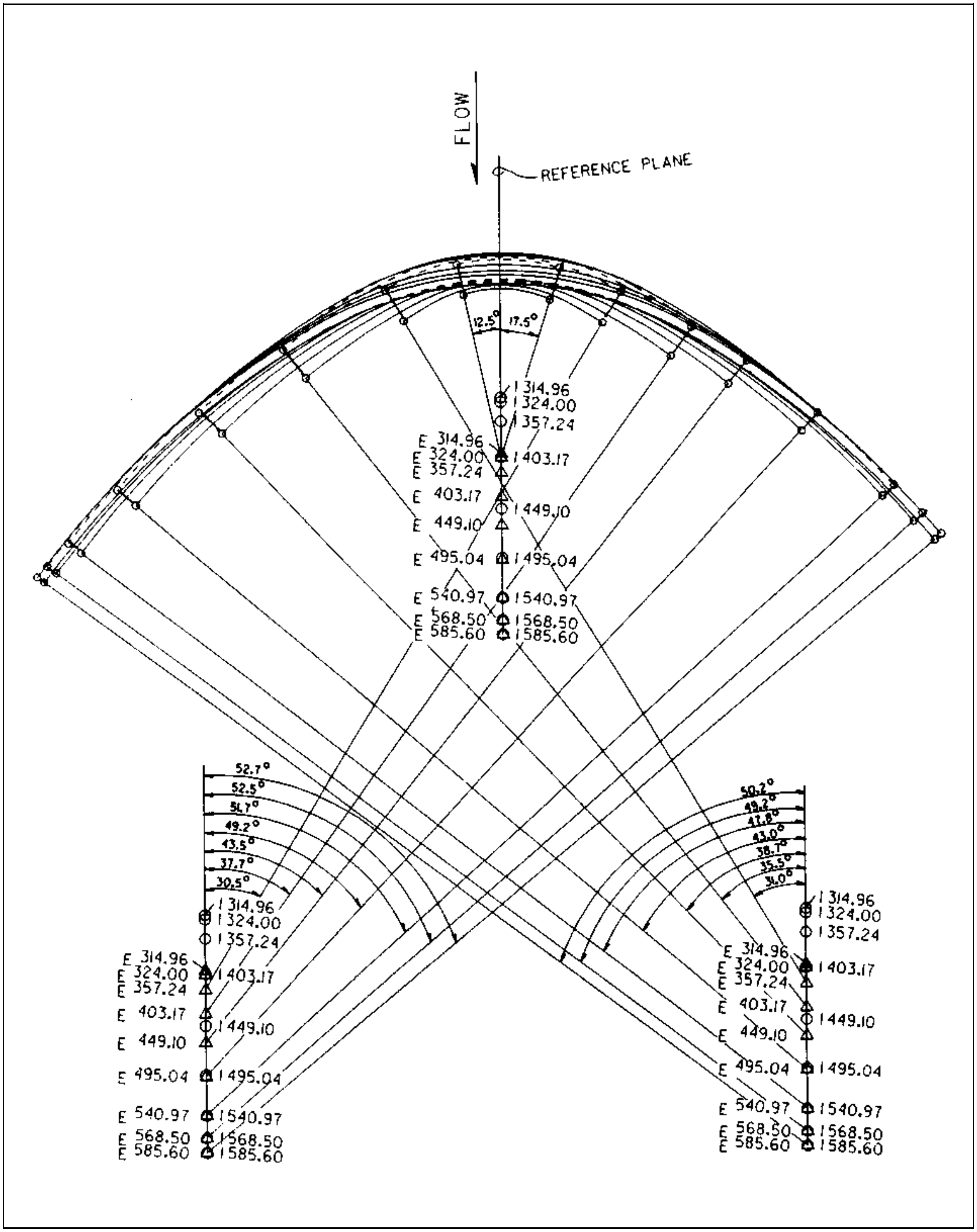

Figure 1-5. Plan of a three-centered variable-thickness arch dam

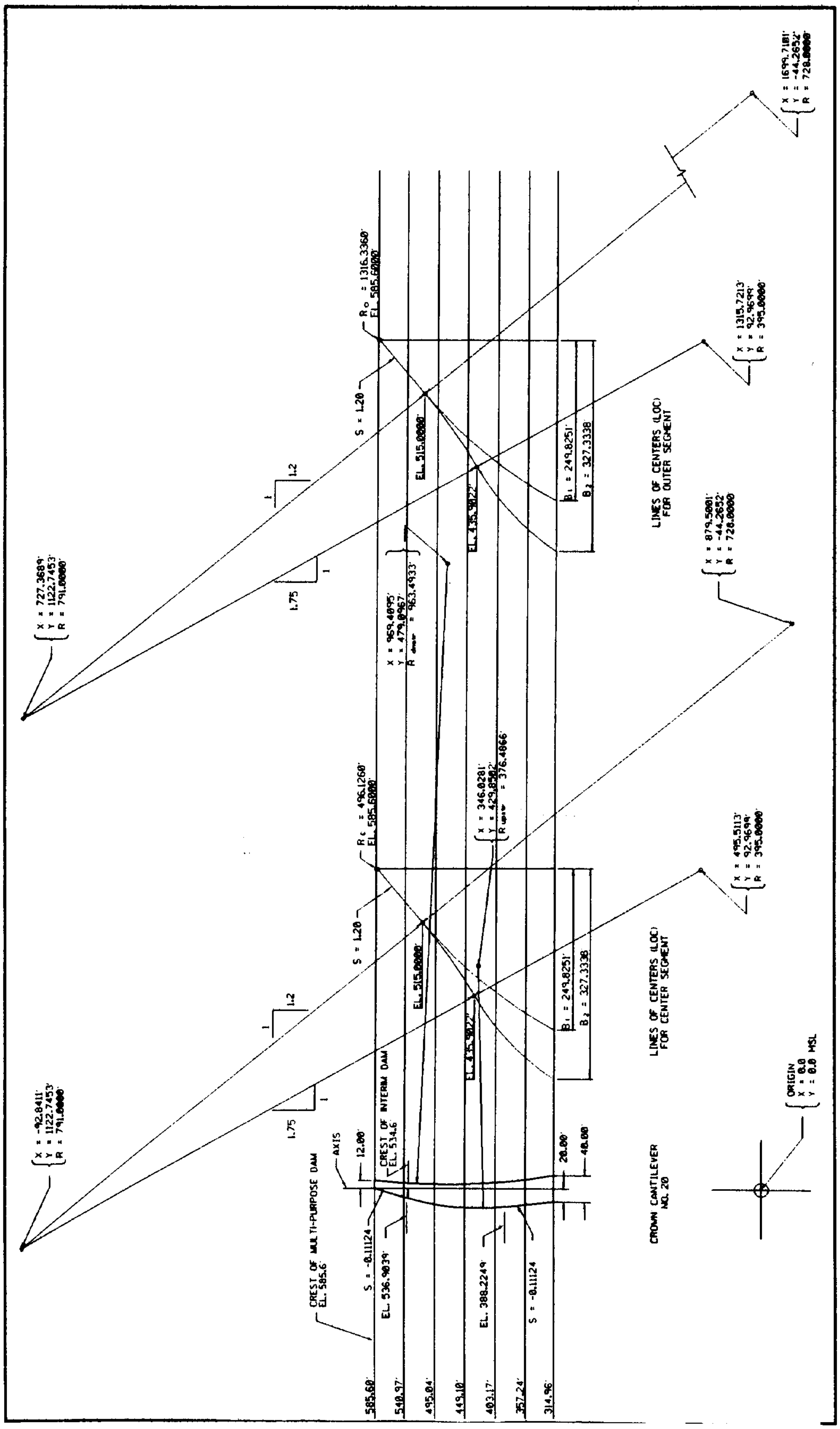

Figure 1-6. Section along reference plane of plane shown in Figure 1-5

of centers are used to describe a three-centered arch dam as shown in Figure 1-5.

k. Crown Cantilever. The crown cantilever is defined as the maximum height vertical cantilever and is usually located in the streambed. It is directed radially toward the axis center. The crown cantilever and the arch crowns are at the same location on symmetrical arch dams. On nonsymmetrical arch dams, the arch crowns will be offset toward the longer side. Maximum radial deflections will occur at the crown cantilever of symmetrical dams but generally between the crown cantilever and arch crowns on nonsymmetrical arch dams.

l. Single Curvature. Single-curvature arch dams are curved in plan only. Vertical sections, or cantilevers, have vertical or straight sloped faces, or may also be curved with the limitation that no concrete overhangs the concrete below. These types of shapes were common prior to 1950.

m. Double Curvature. Double-curvature arch dams means the dam is curved in plan and elevation as shown in Figure 1-7. This type of dam utilizes the concrete weight to greater advantage than single-curvature arch dams. Consequently, less concrete is needed resulting in a thinner, more efficient dam.

n. Overhang. Overhang refers to the concrete on the downstream face where the upper portion overhangs the lower portion. Overhang is most at the crown cantilever, gradually diminishing toward the abutments. The overhanging concrete tends to negate tension on the downstream face in the upper one-quarter caused by reaction of the lightly loaded upper arches.

o. Undercutting. Undercutting refers to the upstream face where the concrete/rock contact undercuts the concrete above it. Undercutting causes the moment from concrete weight to compress the concrete along the heel and tends to negate tension from the reservoir pressure. If an exaggerated undercutting becomes necessary, an imbalance during construction may occur in which case several of the concrete blocks may have to be supported with mass concrete props placed integrally with the blocks. Each prop width is less than the block width to avoid additional arch action. The lowest lift within the prop is painted with a bond breaker to avoid additional cantilever action. Undercutting is most predominant at the base of the crown cantilever. Generally, as the crest length-to-height ratio increases so do the overhang and the undercutting.

p. Symmetrical. In addition to the canyon shapes previously described, the canyon is also described as symmetrical or nonsymmetrical. In general, sites are not absolutely symmetrical but are considered symmetrical if the arch lengths on each side differ by less than about 5 percent between 0.15H and 0.85H. Figure 1-3 shows the plan view of a typical dam in a symmetrical site.

q. Nonsymmetrical. Nonsymmetrical sites result in dams with longer arches on one side of the crown cantilever than the other. Dams for such sites will quite possibly have two reference planes, one for each side but with a common crown cantilever as shown in Figure 1-4. The short side with

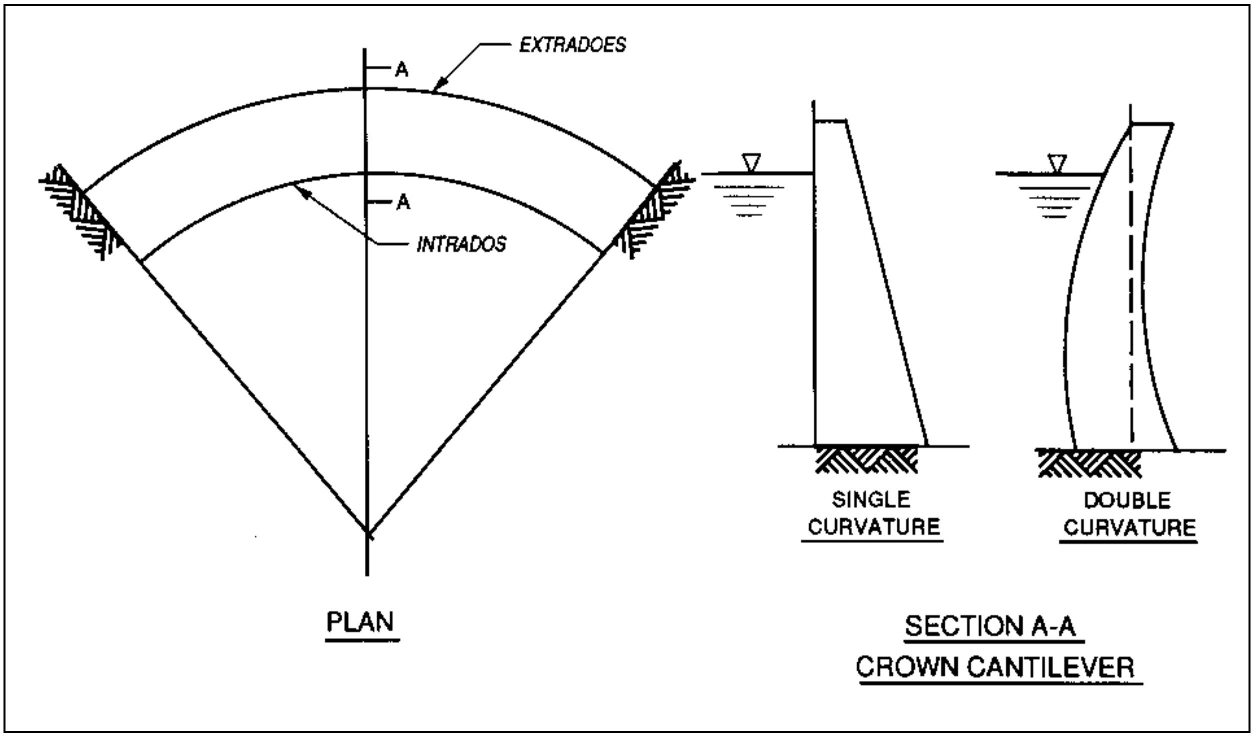

Figure 1-7. Example of single- and double-curvature dams

the steeper-wall canyon will have shorter radii and exhibit more arch action. Whereas the longer side, abutting into the flatter slope, will have less arch action and will be relatively thicker along the abutments. In general, the maximum deflection at each elevation will not occur at the crown cantilever but more toward the midpoint of each arch. A different axis radius for each side will be necessary. To maintain continuity, however, each pair of lines must lie along the reference plane. In some cases the axis radius (R_{axis}) may be different on each side, and the arches may be uniform or variable in thickness. A region of stress concentration is likely to exist in an arch dam having a nonsymmetrical profile. In some cases improvements of a nonsymmetrical layout by one or a combination of the following methods may be warranted: by excavating deeper in appropriate places, by constructing an artificial abutment, or by reorienting and/or relocating the dam.

r. <u>Lines of Centers.</u> A line in space which is the loci of centers for circular arcs is used to describe a face of the dam. For uniform-thickness arches, a single line of centers will describe the extrados and intrados faces. Variable-thickness arches require two lines of centers. Nonsymmetrical sites need one or two lines of centers for each side of the dam. Three-centered arches have three lines of centers as shown in Figure 1-6. It should be noted in Figure 1-6 that the lines of centers for the outer segments are identical and only one pair is shown. Also, in Figure 1-6, arches of variable thickness are used below elevation 515 feet.

s. <u>Constant Center.</u> A constant-center dam has a vertical line at the axis center to describe the center for all arches. All arches are uniform in thickness and the crown cantilever is representative of all vertical sections.

t. Single Center. Single-center constant thickness arches have the same center describing the extrados and the intrados which means all arches are uniform in thickness between abutments. Single-center variable-thickness arches have different centers describing the extrados and the intrados; however, both lie along the reference plane. The lines of centers need not be vertical but must be coplanar with the crown cantilever. This arch shape is applicable to narrow canyon sites such as those with cl:h less than 3:1.

u. Two Centered. In two-centered arches, both planes are coplanar with the crown cantilever. The left plane contains the extrados and intrados lines of centers required to properly shape the left side of the arches as measured from the crown cantilever to the abutments. The right plane of centers contains the extrados and intrados lines of centers for the right-side arches.

v. Three Centered. With three-centered arches, only the center segment is coplanar with the crown cantilever. The center segment and outer segment are coplanar at an angle of compound curvature as measured from the reference plane. Three-centered arches approximate an ellipse. Figure 1-5 shows a typical three-centered arch. A parabola can be approximated by using straight tangents in the outer segment instead of arcs. Three-centered or elliptical arches can be used advantageously in wide-U or V-shaped canyons. Elliptical arches have the inherent characteristics of conforming more nearly to the line of thrust for wide sites than do circular arches. Consequently, the concrete is stressed more uniformly throughout its thickness. Because of the smaller influences from moments, elliptical arches require little, if any, variable thickness.

CHAPTER 2

GENERAL DESIGN CONSIDERATIONS

2-1. Dam Site. Unlike a concrete gravity dam which carries the entire load by its self weight, an arch dam obtains its stability by both the self weight and, to a great extent, by transmitting the imposed loads by arch action into the valley walls. The geometry of the dam site is, therefore, the most basic consideration in the selection of an arch dam. As a general rule, an arch dam requires a site with abutments of sufficient strength to support the arch thrust. On special occasions artificial abutments - thrust blocks - may be used in the absence of suitable abutment(s); see Chapter 3 for additional discussion on thrust blocks.

2-2. Length-Height Ratio. Traditionally, most of the arch dams in the United States have been constructed in canyon sites with length-height ratios of less than 4 to 1. Although the greatest economic advantage may be realized for a length-height ratio of less than 4 to 1, sites with greater ratios should also be given serious consideration. With the present state of the art in arch dam design automation, it is now possible to obtain "optimum design" for sites which would have been considered difficult in the past. An arch dam must be given first consideration for a site with length-height ratio of 3 or less. For sites having length-height ratios between 3 and 6, an arch dam may still provide the most feasible structure depending on the extent of foundation excavation required to reach suitable material. The effect of factors other than length-height ratio becomes much more predominant in the selection process for dam sites with length-height ratios greater than 6. For these sites a careful study must be performed with consideration given to the diversion requirements, availability of construction material, and spillway and outlet works requirements. The results of these studies may prove the arch dam as a viable choice for wider sites.

2-3. Smooth Abutments. The arch dam profile should be made as smooth as practicable. The overall appearance on each abutment should resemble a smooth geometric curve composed of one or two parabolas or hyperbolas. One point of contraflexure in the profile of each abutment will provide for a smooth force distribution along the rock contact. Each original ground surface may have a very irregular profile before excavation, but the prominent points should be removed together with removing weathering to sound rock. Each abutment surface irregularity of peaks and valleys represents points of force concentration at the peaks and correspondingly less force in the valleys. As can be readily surmised, design difficulties lead to structural inefficiencies, more concrete, and increased costs. Thus, it is generally prudent engineering from the beginning to overexcavate the rock and provide for a smooth profile. At the microscale, the abutment should be made smooth, that is, rock knobs remaining after the macroexcavation should, after consensus with the geologist, be removed. Generally, the excavation lines shown in the specifications have tolerances such as ± 1 foot in 20 feet.

2-4. Angle Between Arch and Abutment. Given a geometrically suitable site, another important consideration of an arch dam is the rock contour lines, or the angle which the arches make with the abutment rock contour lines. The angle α in Figure 2-1 should, as a general rule, be greater than 30 degrees to avoid high concentration of shear stresses near the rock surface. Inasmuch as this angle is determined only after the results of the stress analysis are available, the angle β may be used as a guideline during the preparation of the layout. The arches should be arranged so that β is larger than 40 degrees in the upper half. Care must be taken in using these guidelines since the arch thrust, H, is only the tangential component of the total force, and the other two components, vertical and radial, and their respective orientations, must also be examined in the more advanced stages of design. Additionally, the elevation of the arch being investigated should be considered, e.g., an arch located at or near the top of the dam may not be carrying appreciable tangential thrust if the continuity of the arch is broken by an overflow spillway. Observing this criterion - the minimum angle - ensures that there is sufficient rock mass downstream to withstand the applied loads. In addition to this requirement, the directions of joint systems in the rock should be given careful consideration in making the layout to ensure stable abutments under all loading conditions.

2-5. <u>Arch Abutments.</u> Full-radial arch abutments (normal to the axis) are advantageous for good bearing against the rock. However, where excessive excavation at the extrados would result from the use of full-radial abutments and the rock has the required strength and stability, the abutments may be reduced to half-radial as shown in Figure 2-2a. Where excessive excavation at the intrados would result from the use of full-radial abutments, greater-than-radial abutments may be used as shown in Figure 2-2b. In such cases, shearing resistance should be carefully investigated. Where full-radial arch abutments cannot be used because excessive excavation would result from the use of

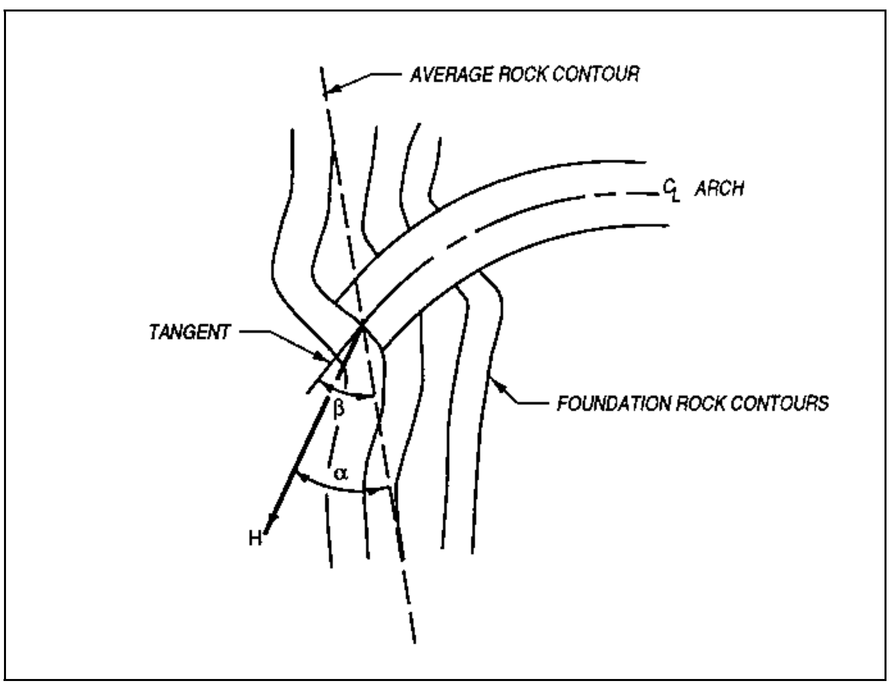

Figure 2-1. Angle between arch thrust and rock contours

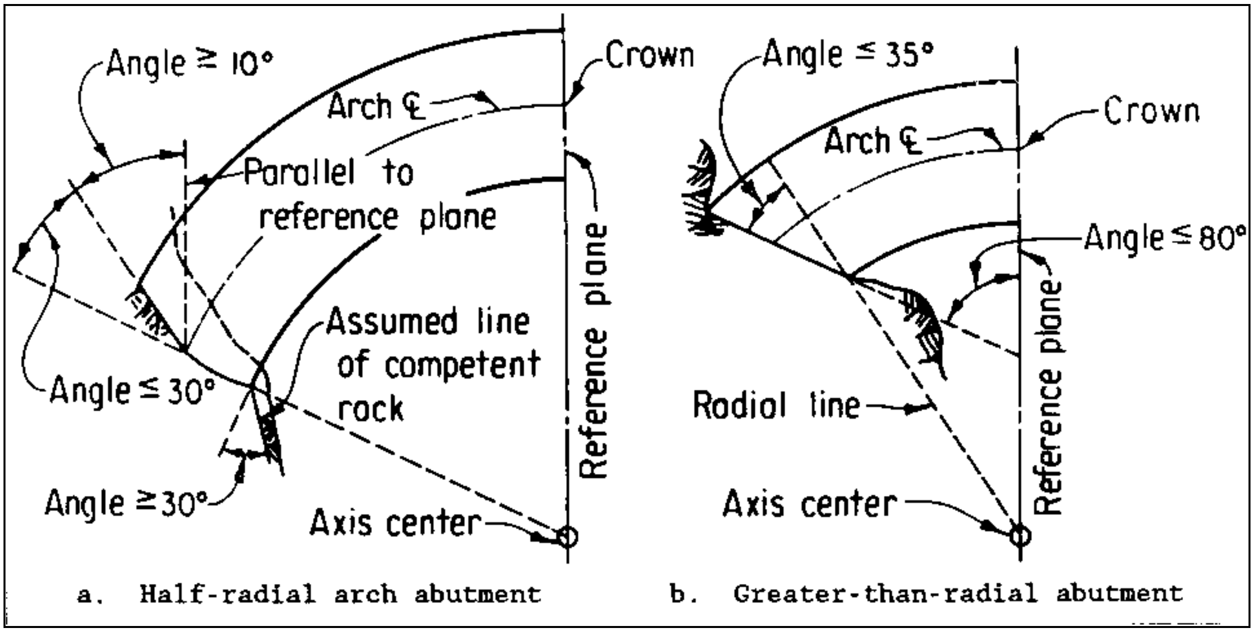

Figure 2-2. Arch abutment types

either of the two shapes mentioned, special studies may be made for determining the possible use of other shapes having a minimum excavation. These special studies would determine to what extent the arch abutment could vary from the full-radial and still fulfill all requirements for stability and stress distribution.

2-6. Foundation. An arch dam requires a competent rock foundation of sufficient strength to withstand the imposed loads from the dam and the reservoir. Inasmuch as the loads are transmitted to the foundation along the entire dam-foundation contact area, the abutment must meet the same minimum foundation requirements as that for the deepest part of the dam, commensurate with the magnitude of resultant forces at a given arch elevation. Because of its small dam-foundation contact area, as compared to other types of dams, an arch dam exerts a larger bearing pressure on the foundation. For the purpose of site selection, a foundation with a compressive strength sufficient to carry the load from a gravity dam would also be satisfactory for an arch dam, recognizing that very seldom are foundations made up of a single type of rock of uniform strength and that this is only an average "effective" value for the entire foundation. Arch dams are capable of spanning weak zones of foundation, and the presence of faults and shears does not appreciably affect the stresses in the dam provided that the thickness of a weak zone is no more than about one times the base thickness of the dam. A description of the treatment of these faults and shear zones is discussed in paragraph 3-5.

2-7. Foundation Deformation Modulus. Deformation behavior of the foundation has a direct effect on the stresses within the dam. Lower values of foundation deformation modulus, i.e., a more yielding foundation, reduce the tension at the base of the dam along the foundation and, conversely, a foundation with high- deformation modulus values results in higher tensile stresses along the base. It is, therefore, important to determine the deformation modulus of the foundation at the earliest stage of design. This information becomes more critical when there are indications that the deformation modulus for one abutment may be drastically different than for the other abutment. Having this knowledge at early stages of design, the structural designer can shape the dam properly so that excessive stresses are avoided. A foundation should not be considered inadequate solely because of low values of deformation moduli. Foundation grouting may improve the deformation behavior of the rock mass and should be considered in determining the deformation moduli used in the design of the dam. When deformation values smaller than 500,000 pounds per square inch (psi) are present, the question of how much a grouting program can improve the foundation becomes critical, and a thorough stress analysis should be performed using a reasonable range of deformation moduli. The design is acceptable if the dam stresses are within allowable stresses under all assumed conditions.

2-8. Effect of Overflow Spillway. If an overflow type of spillway is used and is located near the center of the dam, no arch action is considered above the crest elevation of the spillway. If a spillway is located near one side of the dam, there may be some arch action above the crest elevation of the spillway. In either case, the upper portion of the dam above the spillway crest must be designed to withstand the effects of the loading imposed above the crest by water pressure, concrete mass, temperature, and earthquake.

CHAPTER 3

SPILLWAYS, OUTLET WORKS AND APPURTENANCES, AND RESTITUTION CONCRETE

3-1. Introduction. This chapter describes the influence of voids through the arch dam and structural additions outside the theoretical limits of the arch dam. Voids through the dam in the radial direction are spillways, access adits, and outlet works, in the tangential direction are adits, galleries, and tunnels, and in the vertical direction are stairway wells and elevator shafts. Often associated with spillways and outlet works are blockouts or chambers for gate structures. External structures are restitution concrete which includes thrust blocks, pads, pulvino, socle, or other dental type concrete, spillway flip buckets on the downstream face, and corbels on the upstream face.

3-2. Spillways. Numerous types of spillways are associated with arch dams. Each is a function of the project purpose, i.e., storage or detention, or to bypass flood flows or flows that exceed diversion needs. Spillways for concrete dams may be considered attached or detached.

a. Attached Spillways. Attached spillways are through the crest or through the dam. Through-the-crest spillways have a free fall which is controlled or uncontrolled; OG (ogee) types are shaped to optimize the nappe. In general, the usual crest spillway will be constructed as a notch at the crest. The spillway notch can be located either over the streambed or along one or both abutments as shown in Figure 3-1. A spillway opening can also be placed below the crest through the dam. Similar to the notch spillway, this opening can be located either over the streambed, as shown in Figure 3-2, or near one or both abutments. The location through the dam, whether at the crest or below the crest, is always a compromise between hydraulic, geotechnical, and structural considerations. Impact of the jets on the foundation rock may require treatment to avoid eroding the foundation. Spillways through the dam are located sufficiently below the crest so that effective arch action exists above and below the spillway openings.

(1) Spillway at Crest. With this alignment, the spillway crest, piers, and flipbucket are designed to align the flow with the streambed to cause minimal possible bank erosion and/or to require minimal subsequent beneficiation. However, the notch reduces the arch action by the depth of the notch, i.e., the vertical distance between the dam crest and spillway crest which is normally pure arch restraint is nullified and replaced with cantilever action. To accommodate this reduced stiffness, additional concrete must be added below the spillway crest or the entire arch dam must be reshaped, thus complicating the geometry. Not that this is detrimental, but arch dam shapes work more efficiently when kept simple and smooth in both plan and elevation. Moving the spillway notch to either abutment as shown in Figure 3-1 or splitting the spillway crest length and locating half along each abutment will restore most of the arch action to the dam crest. Spillway notches through the crest near abutments interrupt arch action locally, but not significantly, as can be shown in numerous numerical analysis and scale model studies. The effect of abutment spillways is structurally less distressful on arch dams in wide

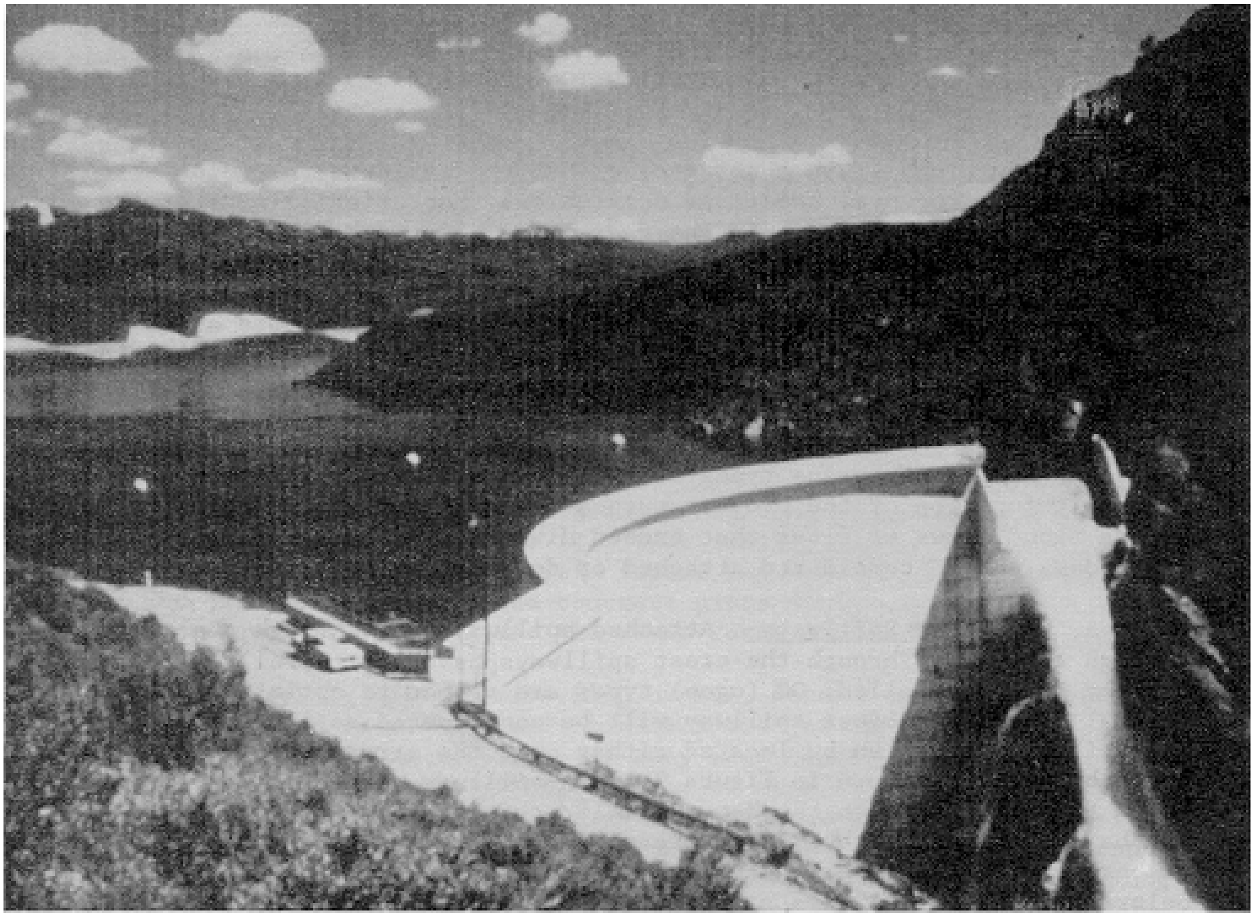

Figure 3-1. East Canyon Dam with spillway notch near left abutment (USBR)

valleys where the top arch is long compared to the structural height, such as a 5:1 crest length-to-height ratio, or in canyons where the climate fluctuates excessively (±50 °F) between summer and winter. In this latter case, winter temperature loads generally cause tensile stresses on both faces near the crest abutment, where the dam is thinnest and responds more quickly and dramatically than thicker sections. Thus, locating the spillway notches along the abutments is a natural structural location. The effects of a center notch, in addition to reducing arch action, are to require that concrete above the spillway crest support the reservoir load by cantilever action. Consequently, design of the vertical section must not only account for stresses from dead load and reservoir but meet stability requirements for shear. Temperature load in this portion of the dam is usually omitted from structural analyses. Earthquake loads also can become a problem and must be considered. On certain arch dams where the spillway width is a small proportion of the total crest length, some arch action will occur in the adjacent curved sections that will improve resistance to flood loads. Usually the beginning of this arch action is about the notch depth away from the pier. The more recent arch dams are thin efficient structures that make flood loads above the spillway crest of greater concern.

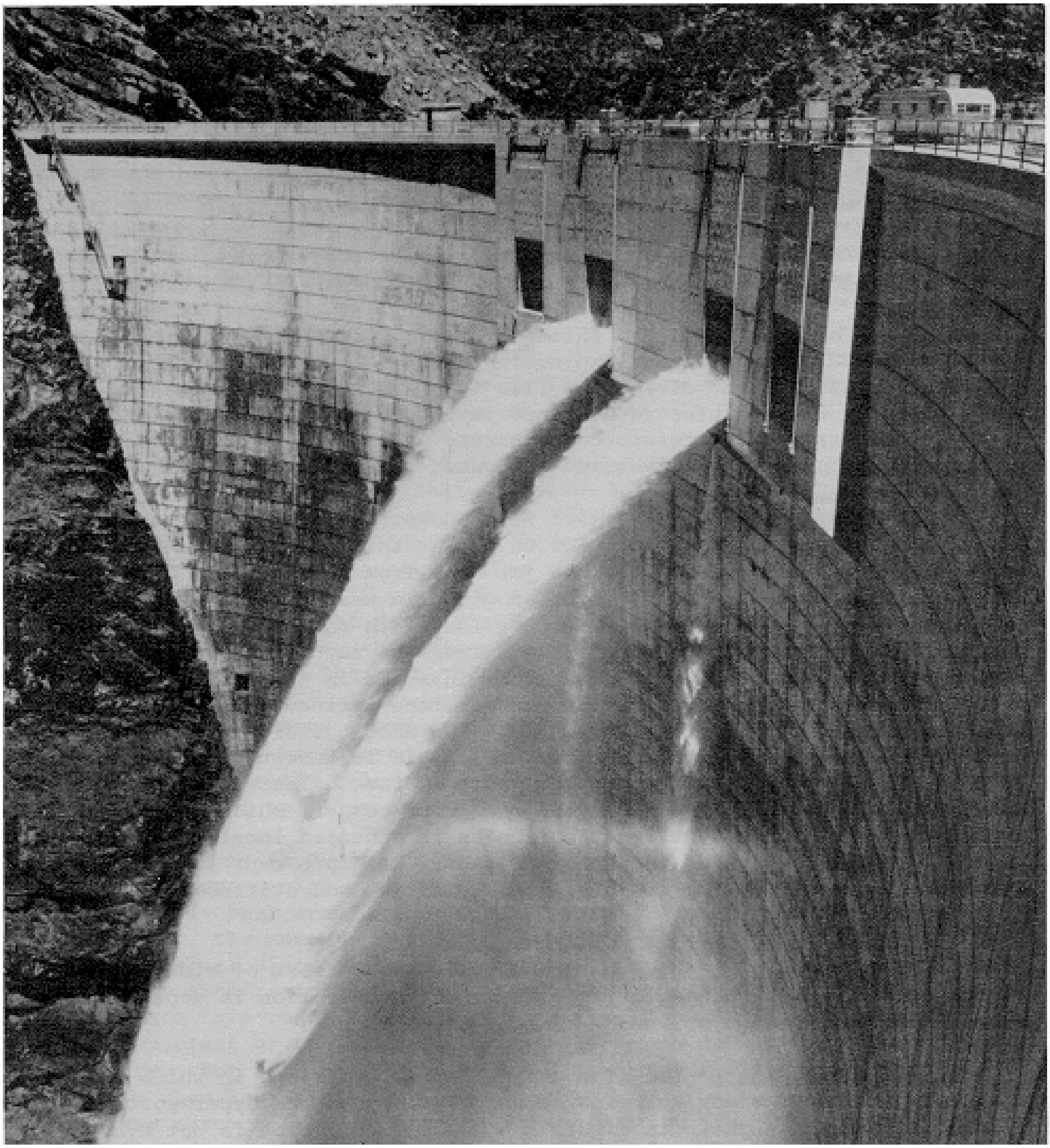

Figure 3-2. Through spillway below crest on Morrow Point Dam (USBR)

(2) Spillways Below Crest (thru Spillways). Spillways are constructed through the arch dam at some optimal distance below the dam crest to reduce the plunge and provide for additional discharge. The spillway may be visualized as multiple orifices, either round or rectangular, and controlled with some type of gates. The set of openings either may be centered over the streambed as shown in Figure 3-2 or split toward either or both abutments. The set is surrounded by mass concrete and locally reinforced to preserve, for the analyses, the assumption of a homogenous and monolithic structure. With this in mind, local reinforcement and/or added mass concrete must be designed so that the dam as a whole is not affected by the existence of the spillway. To minimize disruption of the flow of forces within the dam, the several openings should be aligned with the major principal compressive stresses resulting from the most frequent loading combination. Around the abutment, the major principal stresses on the downstream face are generally normal to the abutment. This alignment would tend to stagger the openings, thus creating design difficulties. In practice, however, all openings are aligned at the same elevation and oriented radially through the dam. If necessary, each orifice may be directed at a predetermined nonradial angle to converge the flows for energy dissipation or to direct the flow to a smaller impact area such as a stilling basin or a reinforced concrete impact pad. Between each orifice, within a set, are normal reinforced concrete piers designed to support the gravity load above the spillway and the water force on the gates.

(3) Flip Bucket. The massive flip bucket, depending on site conditions, may be located near the crest to direct the jet impact near the dam toe or farther down the face to flip the jet away from the toe. In either case, the supporting structure is constructed of solid mass concrete generally with vertical sides. By judiciously limiting its width and height, the supporting structure may be designed not to add stiffness to any of the arches or cantilevers. To assure this result, mastic is inserted in the contraction joints to the theoretical limits of the downstream face defined before the flip bucket was added as shown in Figure 3-3. The mastic disrupts any arch action that might develop. For the same reason, mastic is inserted during construction in the OG corbel overhang on the upstream face. These features protect the smooth flow of stresses and avoid reentrant corners which may precipitate cracking or spalling. Cantilever stiffness is enhanced locally but not enough to cause redistribution of the applied loads. Reinforcement in the supporting structure and accompanying training walls will not add stiffness to the arch dam.

b. Detached Spillways. Detached spillways consist of side channel, chute, tunnel, and morning glory spillways. The selection is dependent upon site conditions.

(1) Side Channel. The side channel spillway is one in which the control weir is placed along the side of and parallel to the upper portion of the discharge channel as shown in Figure 3-4. While this type is not hydraulically efficient nor inexpensive, it is used where a long overflow crest is desired to limit the surcharge head, where the abutments are steep, or where the control must be connected to a narrow channel or tunnel. Consequently, by being entirely upstream from the arch dam, the spillway causes no interference with the dam and only has a limited effect on the foundation. Similarly, along the upstream abutment, any spillway interference is mitigated by the usual low stresses in the foundation caused by the dam loads. Usually,

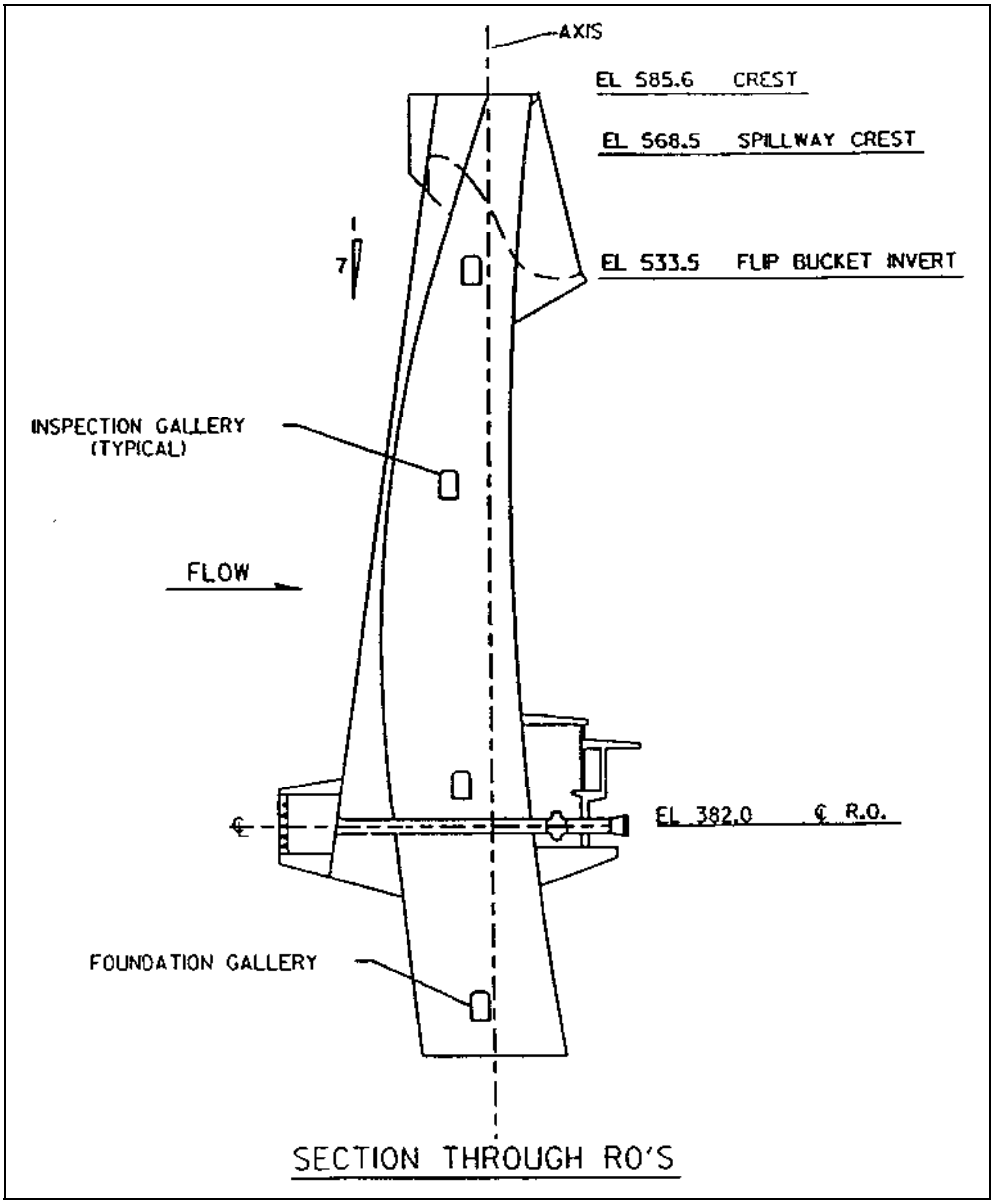

Figure 3-3. Typical section through spillway of a dam

stresses near the crest abutment are much less than the maximum allowable or, quite possibly, are tensile stresses.

(2) Chute Spillway. Chute spillways shown in Figure 3-5 convey discharge from the reservoir to the downstream river level through an open channel placed either along a dam abutment or through a saddle. In either case, the chute is not only removed from the main dam, but the initial slope by being flat isolates the remaining chute from the eventual stressed foundation rock.

(3) Tunnel Spillway. Tunnel spillways convey the discharge around the dam and consist of a vertical or inclined shaft, a large radius elbow, and a

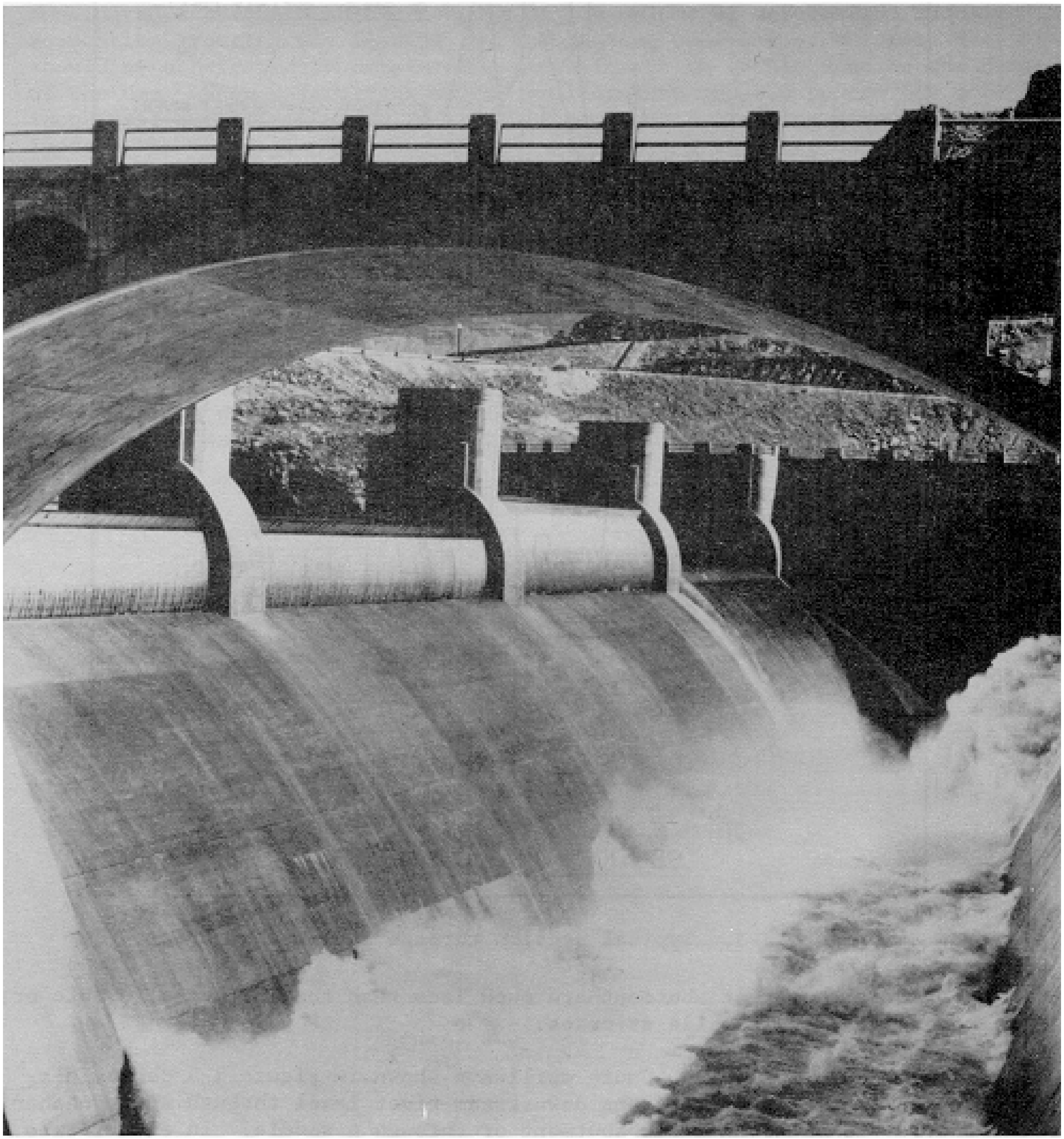

Figure 3-4. Side channel spillway at Hoover Dam (USBR)

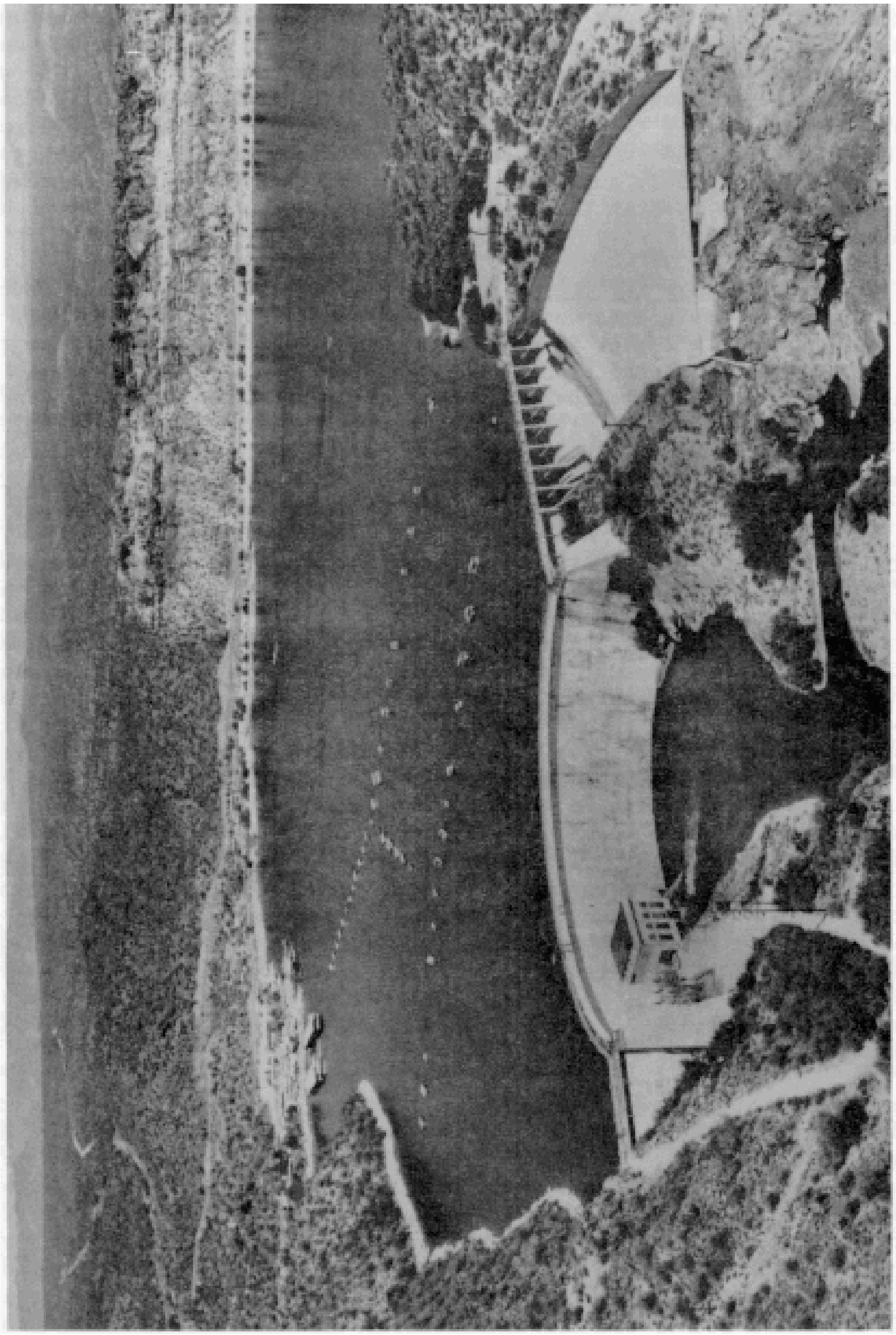

Figure 3-5. Chute spillway at Stewart Mountain Dam (USSR)

horizontal tunnel at the downstream end. Tunnel spillways may present advantages for damsites in narrow canyons with steep abutments or at sites where there is danger to open channels from snow or rock slides. The tunnel alignment usually traverses the stressed foundation and consequently should be at least one abutment thickness from the concrete to rock contact. Any need for increased spacing would be based on structural height and applied load combinations. Note that these tunnels are designed to flow up to 75 percent full, in which case in situ and superimposed stresses from the dam may govern the design. The corollary is that the tunnel walls must be strong enough to avoid creating a weakness in the foundation and subsequent stability problems with the dam.

(4) Morning Glory Spillway. A morning glory spillway such as that shown in Figure 3-6 is one in which the water enters over a horizontally positioned lip, which is circular in plan, and then flows to the downstream river channel through a horizontal or near horizontal tunnel. A morning glory spillway usually can be used advantageously at damsites in narrow canyons where the abutments rise steeply or where a diversion tunnel is available for use as the downstream leg. If a vertical drop structure is to be located upstream, it should not interfere structurally with the dam. A sloping tunnel offers the same concerns as previously discussed.

3-3. Outlet Works. Outlet works are a combination of structures and equipment required for the safe operation and control of water released from a reservoir to serve downstream needs. Outlet works are usually classified according to their purpose such as river outlets, which serve to regulate flows to the river and control the reservoir elevation, irrigation or municipal water supply outlets, which control the flow of water into a canal, pipeline, or river to satisfy specified needs, or, power outlets, which provide passage of water to turbines for power generation. In general, outlet works do not structurally impact the design of an arch dam as shown in Figure 3-7. The major difficulty may lie in adapting the outlet works to the arch dam, especially a small double-curvature thin arch dam where the midheight thickness may be 25 feet or less. Taller and/or heavier dams will have a significant differential head between the intake and the valve or gate house, and the conduit may have several bends. All of the features can be designed and constructed but not without some compromise. Outlet works should be located away from the abutments to avoid interference with the smooth flow of stresses into the rock and smooth flow of water into the conduit. A nominal distance of 10 diameters will provide sufficient space for convergence of stresses past the conduit.

a. Intake Structures. Intake structures, in addition to forming the entrance into the outlet works, may accommodate control devices and the necessary auxiliary appurtenances such as trashracks, fish screens, and bypass devices (Figure 3-8). An intake structure usually consists of a submerged structure on the upstream face or an intake tower in the reservoir. Vertical curvature on the upstream face may result in more than usual massive concrete components as shown in Figure 3-9 to provide a straight track for the stop logs or bulkhead gate. A compromise to relieve some of the massiveness is to recess portions of the track into the dam face. This recess which resembles a rectangular notch in plan reduces the stiffness of those arches involved not only at the notch but for some lateral distance, depending on the notch depth.

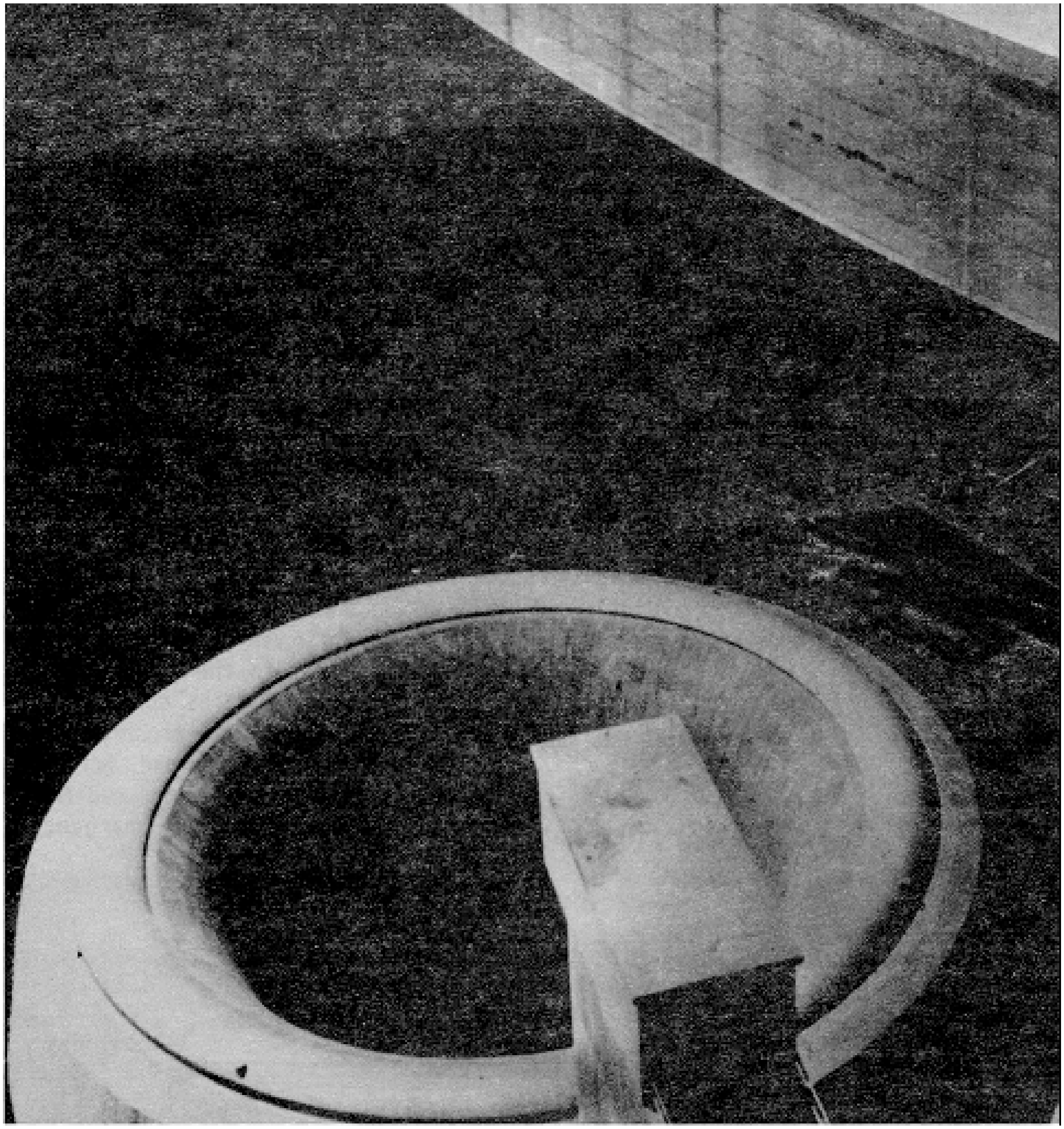

Figure 3-6. Morning glory spillway at Hungry Horse Dam. Note upstream face of dam in upper right (USBR)

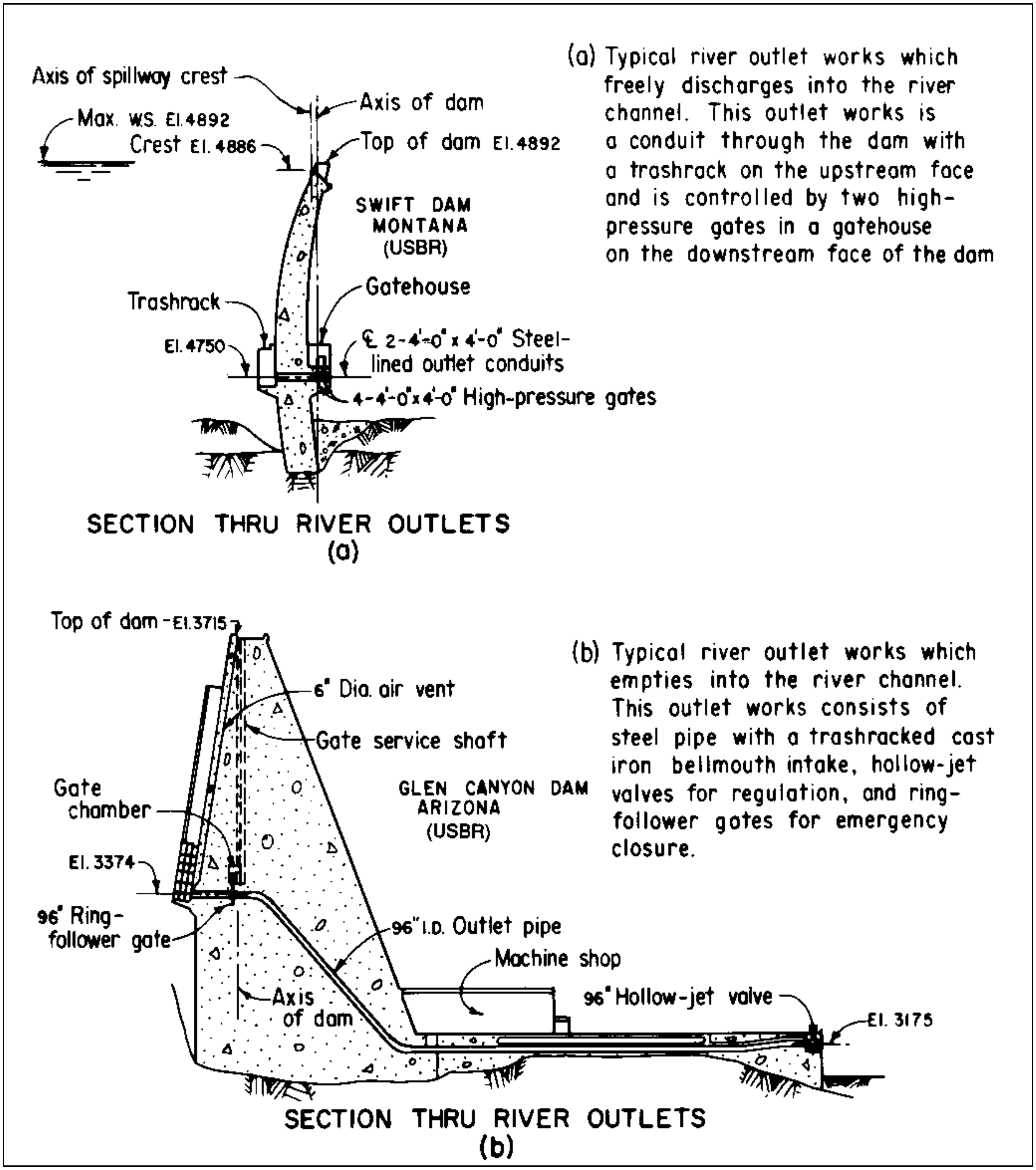

Figure 3-7. Typical river outlet works without a stilling basin

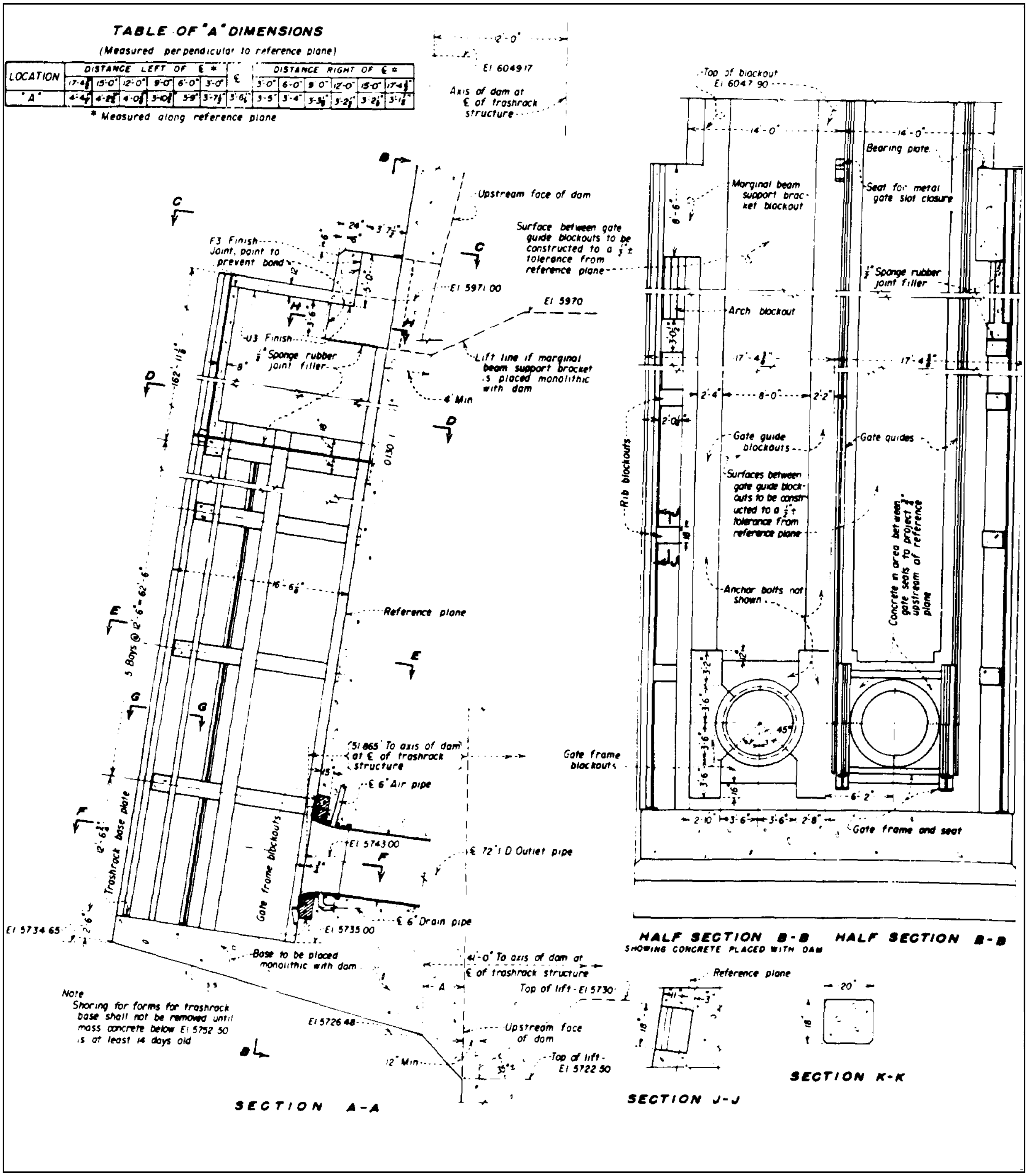

Figure 3-8. Typical river outlets trashrack structure (USBR)

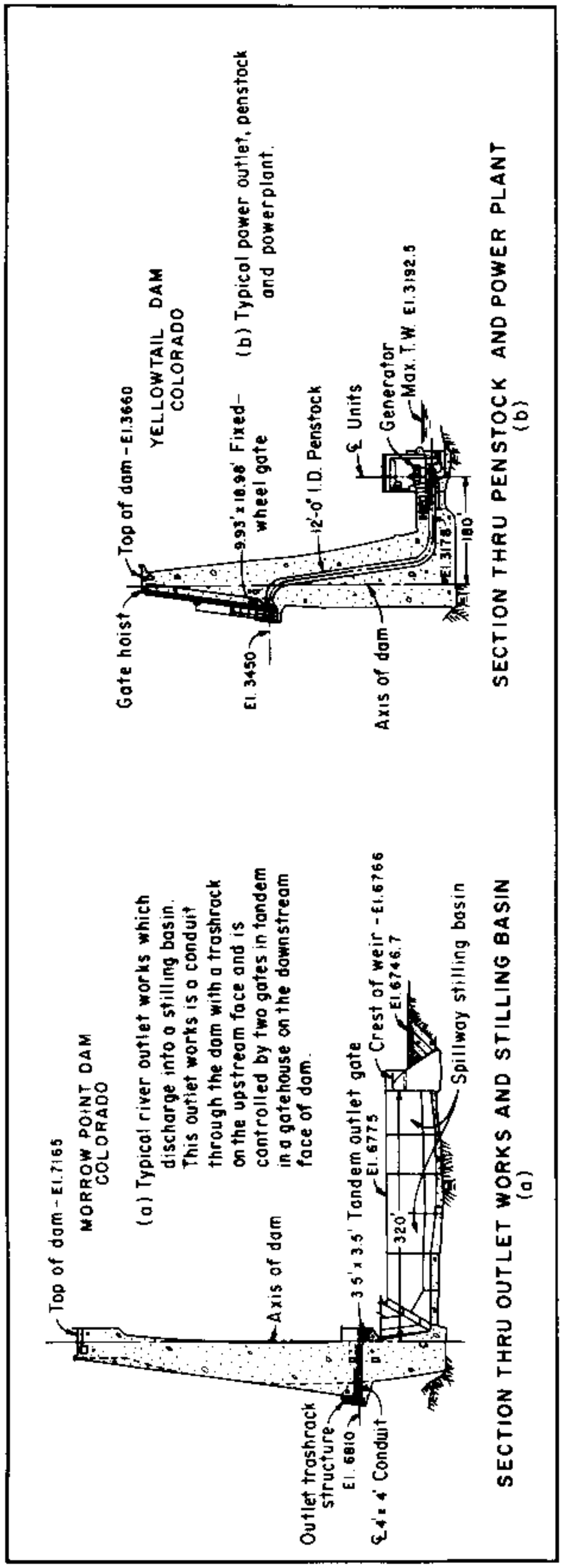

Figure 3-9. Typical river outlet works and power outlet (USBR)

b. Conduit. The outlet works conduit through a concrete dam may be lined or unlined, as in a power outlet, (Figure 3-10) but when the conduit is lined it may be assumed that a portion of the stress is being taken by the liner and not all is being transferred to the surrounding concrete. The temperature differential between the cool water passing through the conduit and the warmer concrete mass will produce tensile stresses in the concrete immediately adjacent to the conduit. In addition, the bursting effect from hydrostatic pressures will cause tensile stresses at the periphery of the conduit. Such tensile stresses and possible propagation of concrete cracking usually extend only a short distance in from the opening of the conduit. It is common practice to reinforce only the concrete adjacent to the opening. The most useful method for determining the stresses in the concrete surrounding the outlet conduit is the finite element method (FEM) of analysis.

c. Control House. The design of a control house depends upon the location and size of the structure, the operating and control equipment required, and the conditions of operation. The loadings and temperature conditions used in the design should be established to meet any situation which may be expected to occur during construction or during operation. In thin dams, the floor for the house is normally a reinforced concrete haunched slab cantilevered from the downstream face and designed to support the valves and houses. Neither the reinforcement nor the concrete should interfere with structural action of the arches and cantilevers. Emergency gates or valves are used only to completely shut off the flow in the outlet for repair, inspection maintenance or emergency closure. Common fixed wheel gates, such as shown in Figure 3-11, are either at the face or in a slot in the dam.

3-4. Appurtenances.

a. Elevator Tower and Shaft. Elevators are placed in concrete dams to provide access between the top of the dam and the gallery system, equipment and control chambers, and the power plant as shown in Figure 3-12. The elevator structure consists of an elevator shaft that is formed within the mass concrete and a tower at the crest of the dam. The shaft should have connecting adits which provide access into the gallery system and into operation and maintenance chambers. These adits should be located to provide access to the various galleries and to all locations at which monitoring and inspection of the dam or maintenance and control of equipment may be required. Stairways and/or emergency adits to the gallery system should be incorporated between elevator stops to provide an emergency exit such as shown in Figure 3-13. The design of reinforcement around a shaft can be accomplished by the use of finite element studies using the appropriate forces or stresses computed when analyzing the arch dam. In addition, stresses within the dam near the shaft due to temperature and other appropriate loads should be analyzed to determine if tension can develop at the shaft and be of such magnitude that reinforcement would be required. To minimize structural damage to the arches, the shaft should be aligned totally within the mass concrete and centered between the faces. However, the necessarily vertical shaft may not fit inside a thin arch dam. In such a case, the shaft could be moved to the abutment or designed to be entirely outside the dam but attached for vertical stability to the downstream face. This solution may not be esthetically pleasing, but it is functional, because if the shaft emerges through the downstream face it forms a rectangular notch that diverts the smooth flow of arch stresses and reduces arch stiffness.

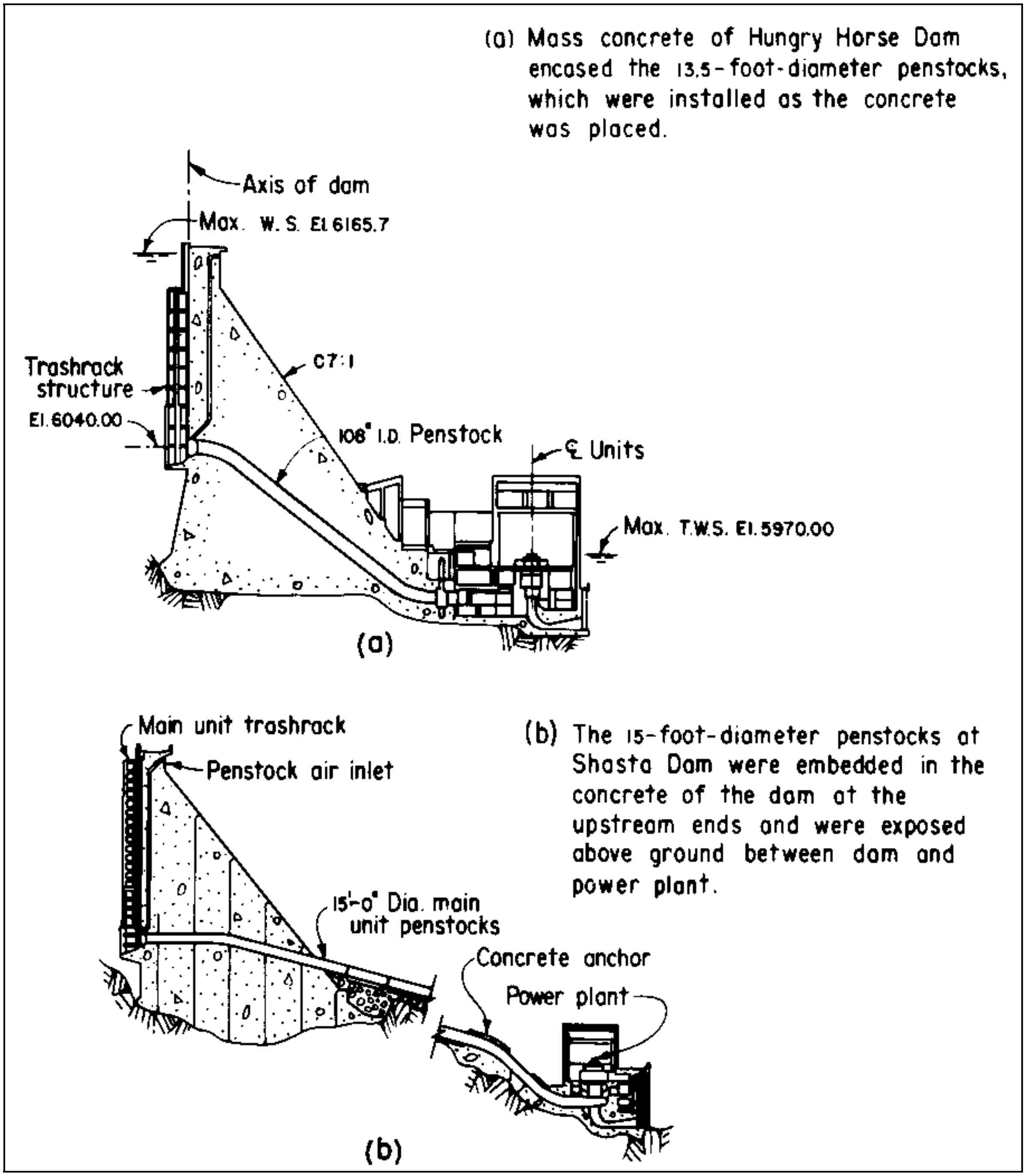

Figure 3-10. Typical penstock installations (USBR)

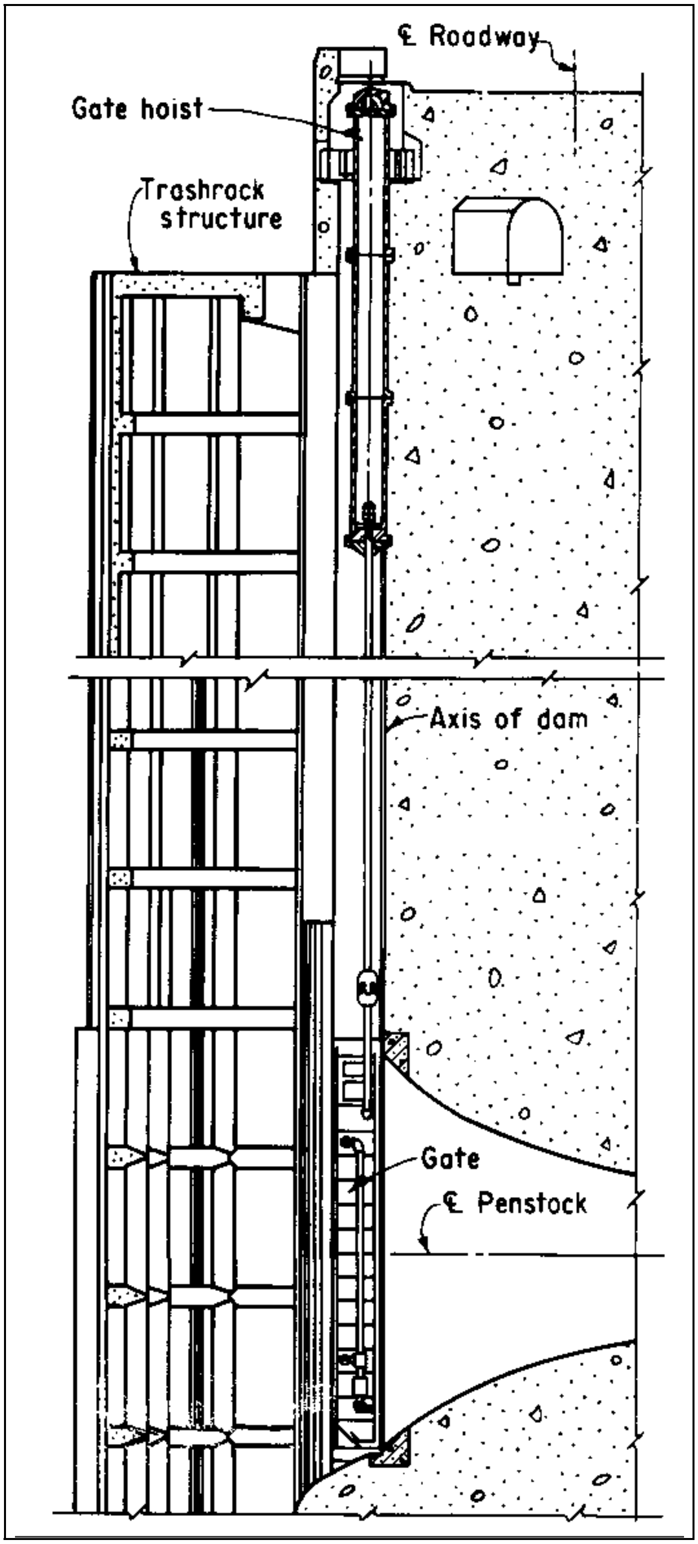

Figure 3-11. Typical fixed wheel gate installation at upstream face of dam (USBR)

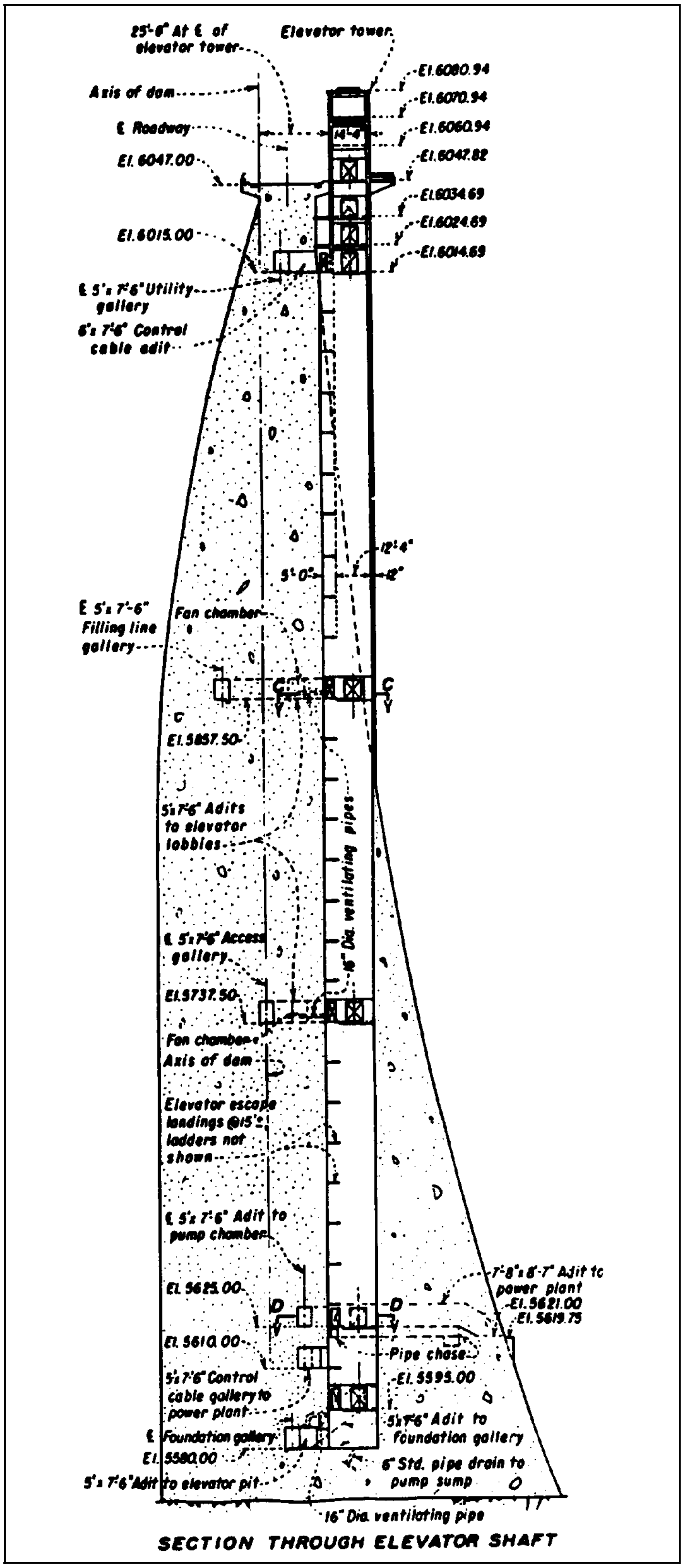

Figure 3-12. Structural and architectural layout of elevator shaft and tower in Flaming Gorge Dam (USBR) (Continued)

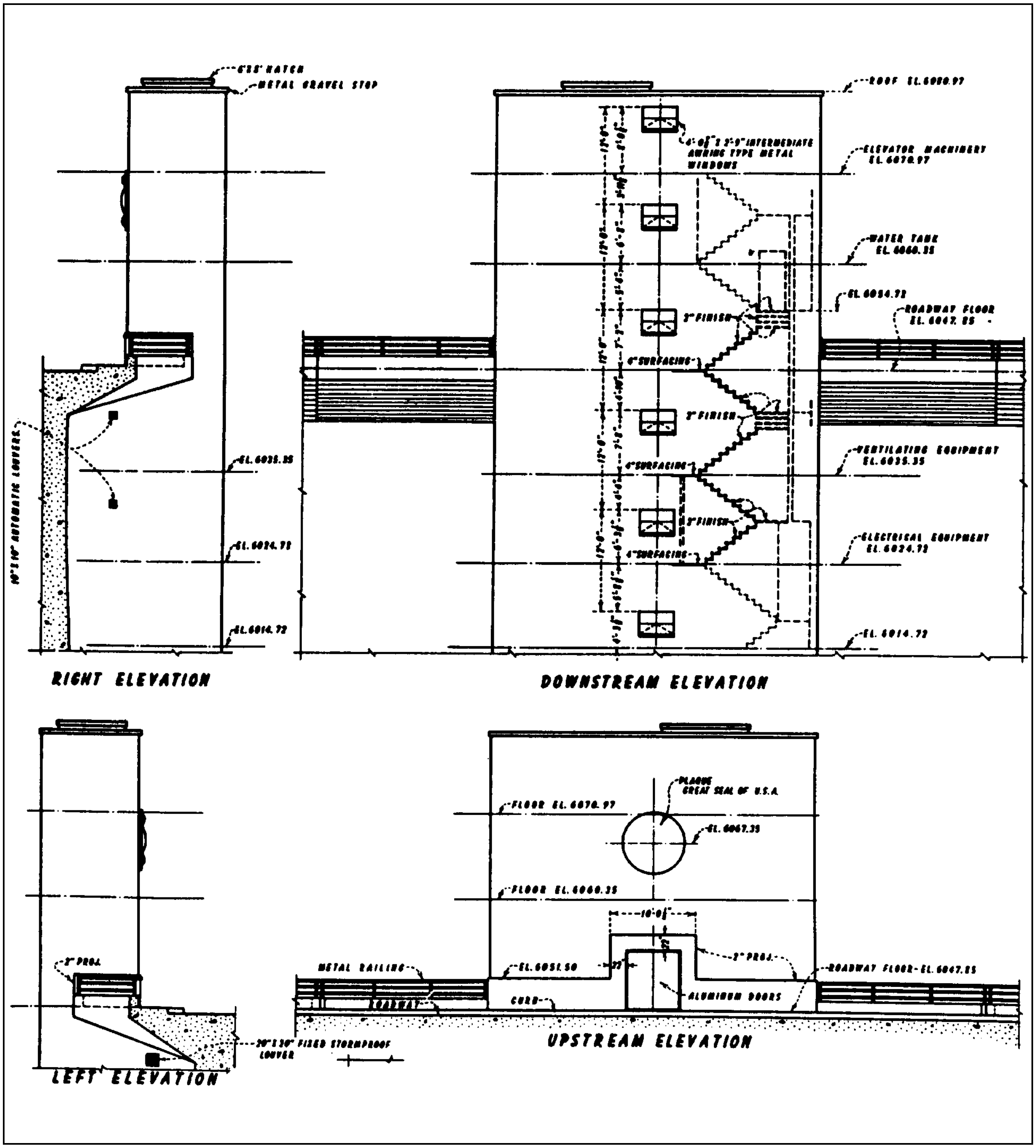

Figure 3-12. (Concluded)

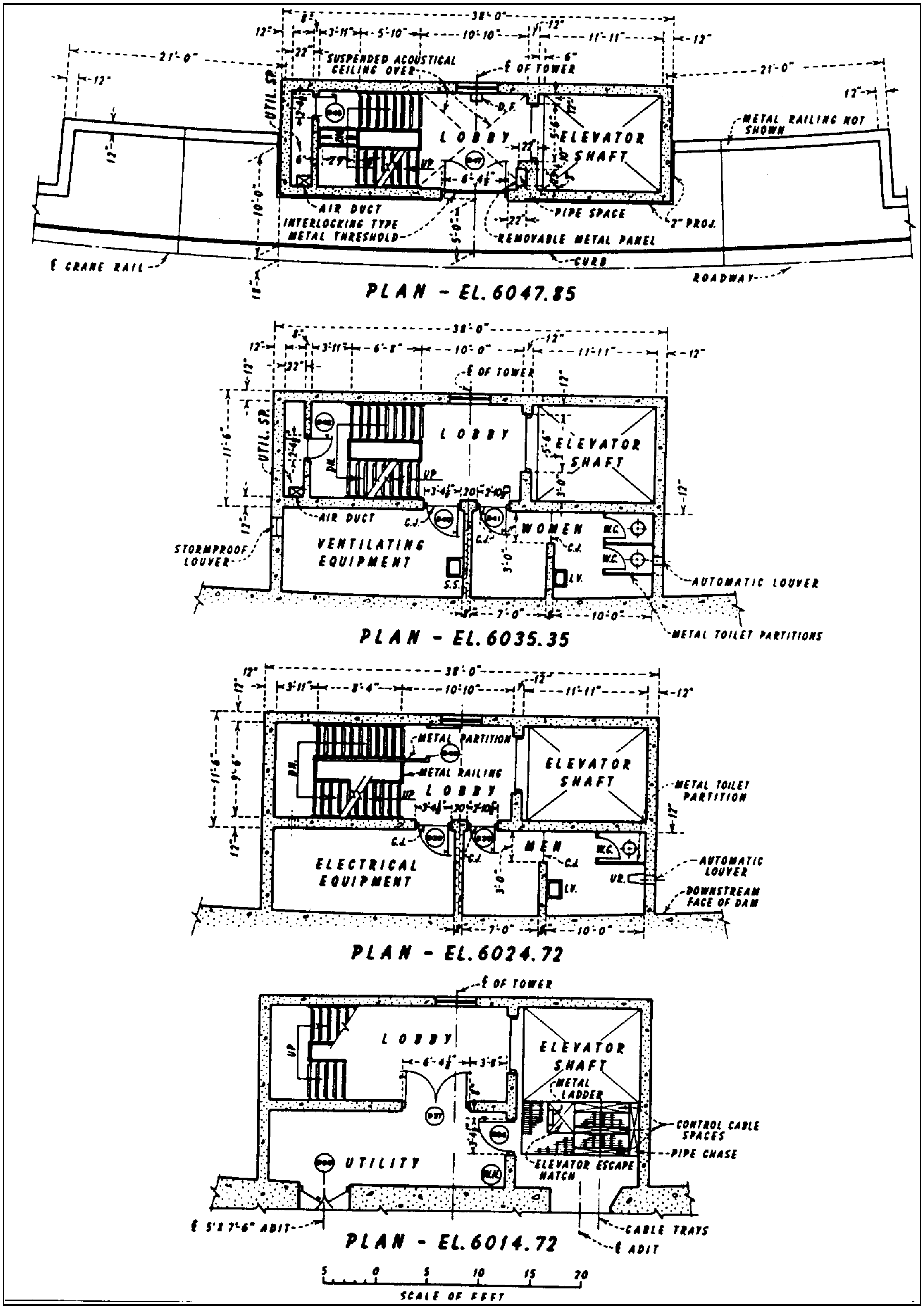

Figure 3-13. Details of layout of elevator shaft and tower in Flaming Gorge Dam (USBR)

b. Bridges. Bridges may be required on the top of the dam to carry a highway over the spillway or to provide roadway access to the top of the dam at some point other than its end. Design criteria for highway bridges usually conform to the standard specifications adopted by the American Association of State Highway Officials and are modified to satisfy local conditions and any particular requirement of the project. Bridges, regardless of how heavy, whether of steel or concrete, or with fixed or pinned connections, are not considered sufficiently strong to transfer arch loads from one mass concrete pier to the other.

c. Top of the Dam. The top of the dam may contain a highway, maintenance road, or walkway such as shown in Figure 3-14. If a roadway is to be built across the dam, the normal top of the dam can be widened with corbels which cantilever the road or walkway out from the upstream or downstream faces of the dam. The width of the roadway on the top of the dam is dependent upon the type and size of roadway, sidewalks, and maintenance and operation space needed to accomplish the tasks required. Parapets or handrails are required both upstream and downstream on the top of the dam and should be designed to meet safety requirements. The minimum height of parapet above the sidewalk should be 3 feet 6 inches. A solid upstream parapet may be used to increase the freeboard if additional height is needed. The design of the reinforcement for the top of the dam involves determining the amount of reinforcement required for the live and dead loads on the roadway cantilevers and any temperature stresses which may develop. Temperature reinforcement required at the top of the dam is dependent upon the configuration and size of the area and the temperature condition which may occur at the site. After the temperature distributions are determined by studies, the temperature stresses that occur can be analyzed by use of the FEM. If a roadway width greater than the theoretical crest thickness of the dam is required, the additional horizontal stiffness of the roadway section may interfere with the arch actions at the top if the dam. To prevent such interference, horizontal contraction joints should be provided in the roadway section with appropriate joint material. The contraction joints should begin at the edge of the roadway and extend to the theoretical limits of the dam face. Live loads such as hoists, cranes, stoplogs, and trucks are not added to the vertical loads when analyzing the arch dam; these loads may weigh less than 10 cubic yards (cu yd) of concrete, an insignificant amount in a concrete dam.

d. Galleries and Adits. A gallery is a formed opening within the dam to provide access into or through the dam. Galleries are either transverse or longitudinal and may be horizontal or on a slope as shown in Figure 3-15. Galleries connecting other galleries or connecting with other features such as power plants, elevators, and pump chambers are called adits. Some of the more common uses or purposes of galleries are to provide a drainageway for water percolating through the upstream face or seeping through the foundation, space for drilling and grouting the foundation, space for headers and equipment used in artificially cooling the concrete blocks and for grouting contraction joints, access to the interior of the structure for observing its behavior, access to and room for mechanical and electrical equipment, access through the dam for control cables and/or power cables, and access routes for visitors, as shown in Figure 3-16. The location and size of a gallery will depend on its intended use or purpose. The size is normally 5 feet wide by 7.5 feet high. In small, thin arch dams, galleries are not used where the radial thickness is less than five times the width. This gives the cantilevers sufficient section

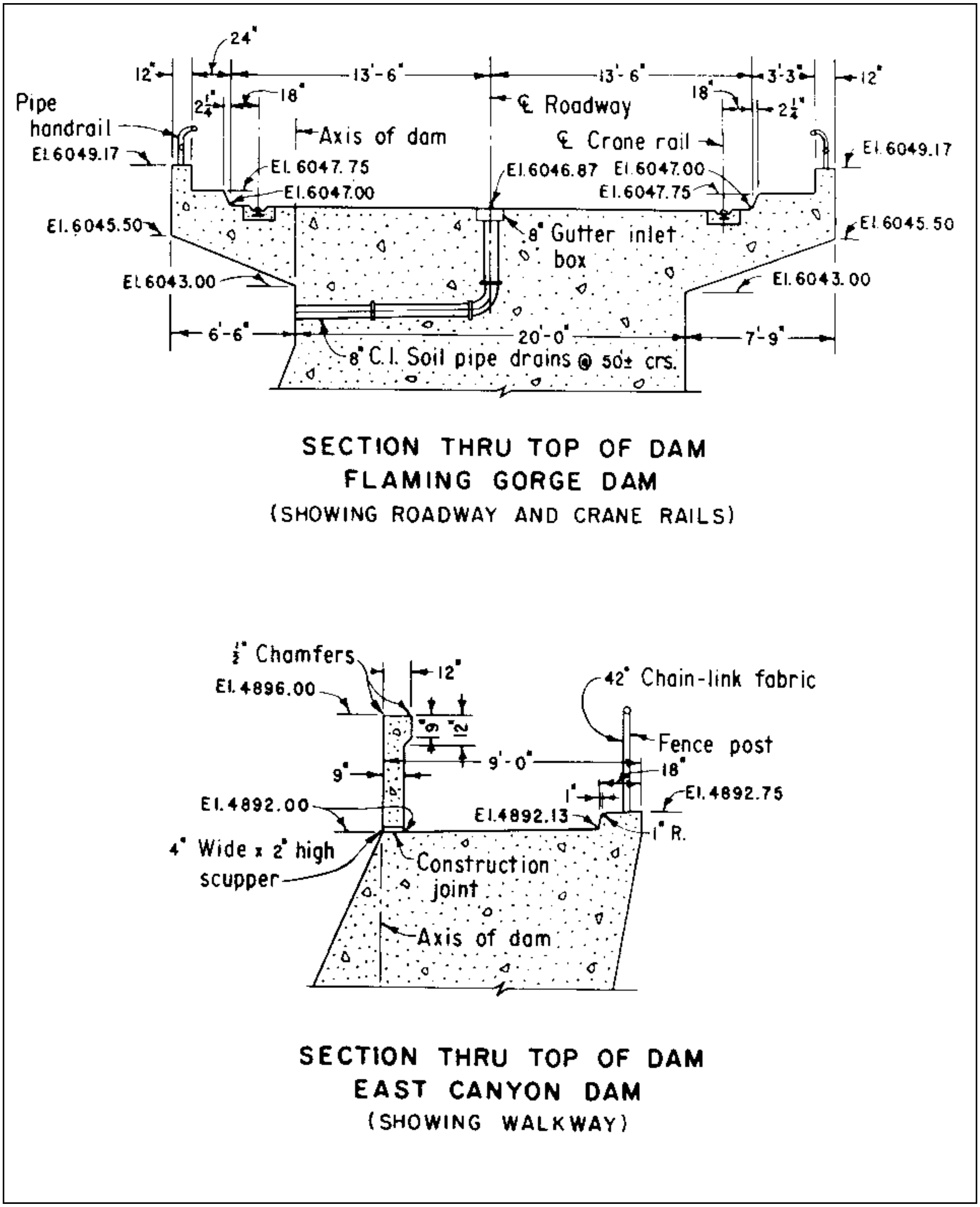

Figure 3-14. Typical sections at top of an arch dam (USBR)

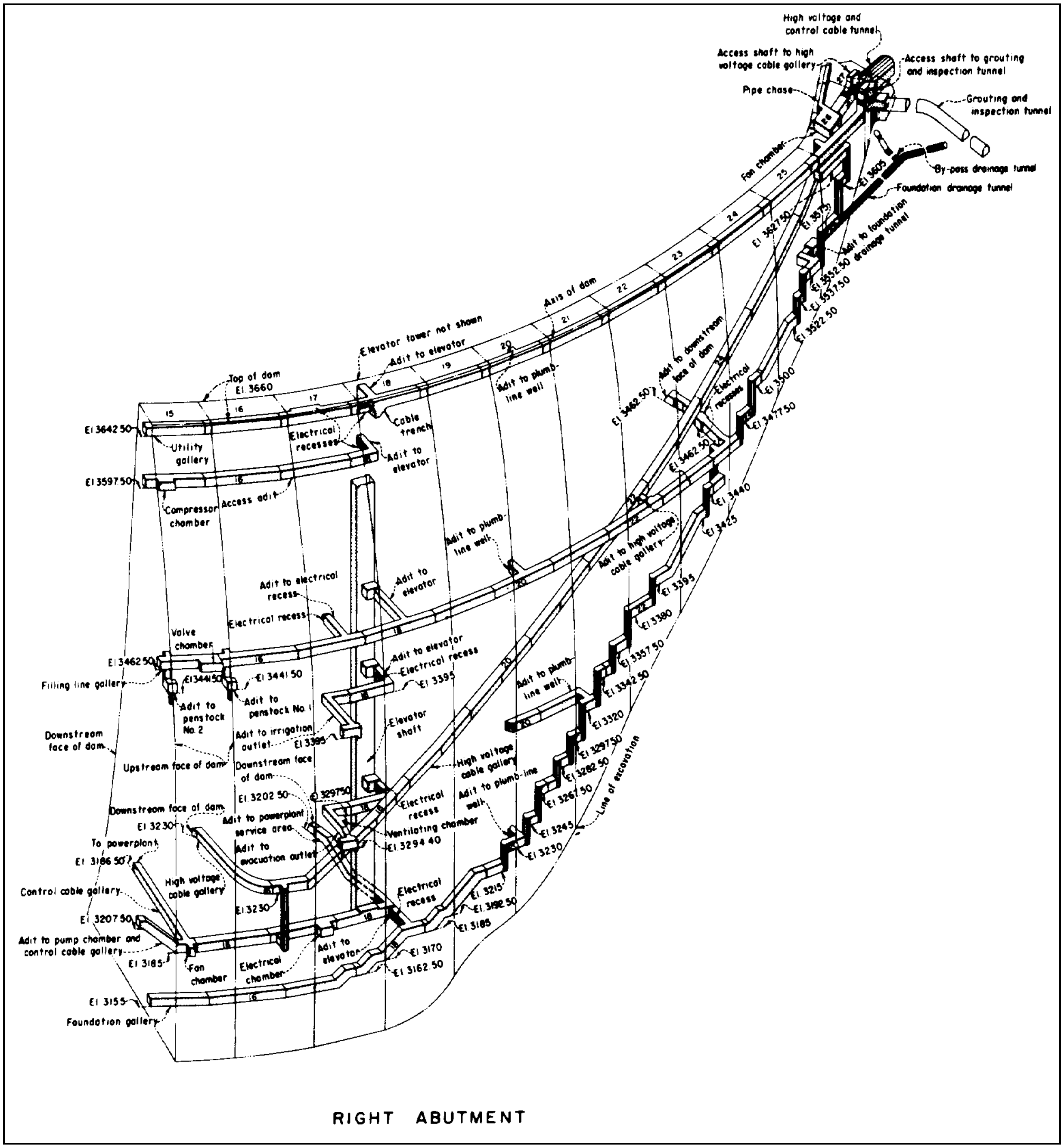

Figure 3-15. Gallery system in right side of Yellowtail Dam (USBR)

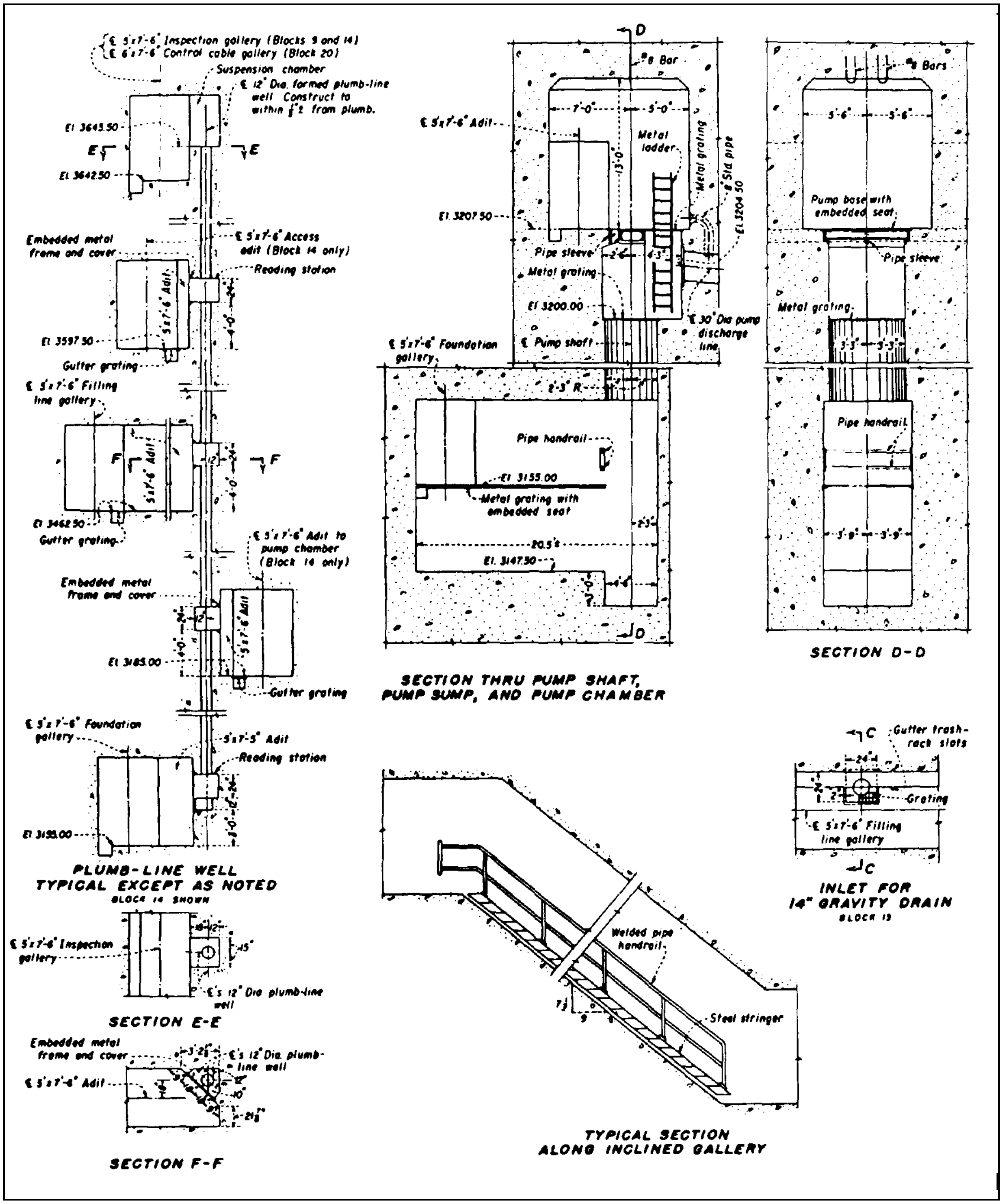

Figure 3-16. Details of galleries and shafts in
Yellowtail Dam (USBR) (Continued)

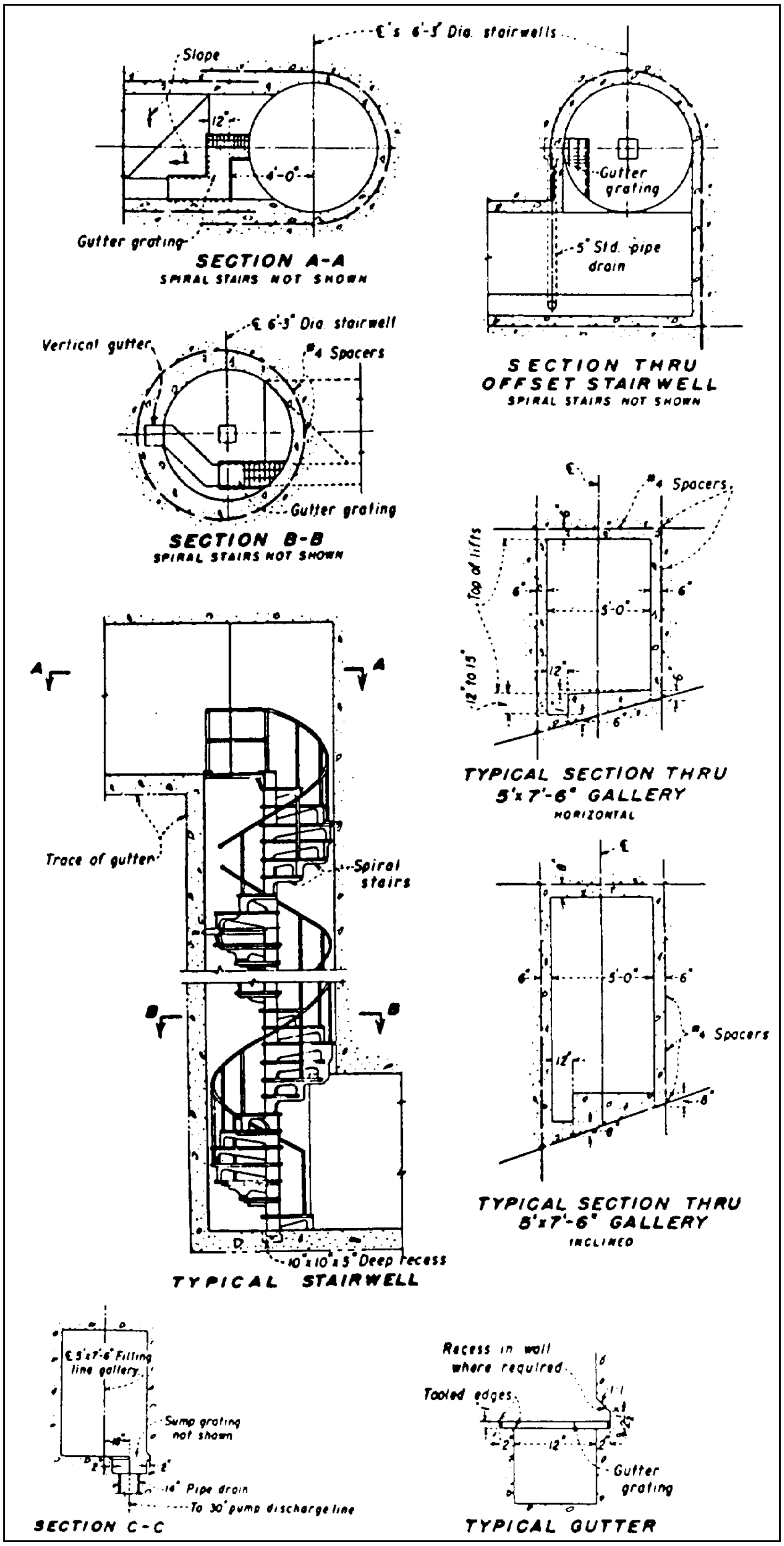

Figure 3-16. (Concluded)

modulus to perform as intended. A distance of two diameters on either side of the gallery provides sufficient thickness to mitigate the high vertical stresses that concentrate along the gallery walls and to develop the necessary section modulus. In thin arch dams, access galleries can be designed smaller than 5 feet wide such as an elliptically shaped gallery 2 feet wide by 8 feet high that contains lighting across the top, a noncorrosive grating for pedestrian traffic, and provides space for drainage.

3-5. Restitution Concrete. Because of topographical and geological features, all damsites are nonsymmetrical and have an irregular profile; however, during the design of an arch dam, a significant saving in keyway excavation may be achieved by building up certain regions of the footprint with mass concrete to form an artificial foundation and provide a smooth perimeter for the dam. At a particular site, restitution concrete may be local "dental" concrete, the more extensive "pad," or "thrust blocks" along the crest. In each case, the longitudinal and transverse shape is different for different design purposes, and, accordingly, restitution concrete may be extensive upstream, downstream, or around the perimeter of the dam. In keeping with the concept of efficiency and economy, each arch dam design should be made as geometrically simple as possible; the optimum is a symmetrical design. With this in mind, restitution concrete can be added to the foundation to smooth the profile and make the site more symmetrical, and/or provide a better distribution of stresses to the foundation. Restitution concrete is added to the rock contact either before or during construction; the concrete mix is the same mass concrete used to construct the dam.

a. Dental Concrete. Dental concrete is used to improve local geological or topographical discontinuities that might adversely affect stability or deformation as shown in Figure 3-17. Discontinuities include joints, seams, faults, and shattered or inferior rock uncovered during exploratory drilling or final excavation that make complete removal impractical. The necessary amount of concrete replacement in these weak geological zones is usually determined from finite element analyses in which geologic properties, geometric limits, and internal and/or external loads are defined. For relatively homogenous rock foundations with only nominal faulting or shearing, the following approximate formulas can be used for determining the depth of dental treatment:

$$d = 0.002bH + 5 \text{ for } H \text{ greater than or equal to 150 feet}$$
$$d = 0.3b + 5 \text{ for } H \text{ less than 150 feet}$$

where

H = height of dam above general foundation level, in feet
b = width of weak zone, in feet, and
d = depth of excavation of weak zone below surface of adjoining sound rock, in feet

b. Pad. A concrete pad is added to the foundation to smooth the arch dam profile, to make the site more symmetrical, to reduce excavation, and/or to provide a better distribution of stresses on the foundation. To smooth the profile, pad concrete is placed around the arch dam perimeter, made irregular due to topography or geology, such as in box canyons or bridging low-strength rock types. Such treatment is not unique to the United States. The

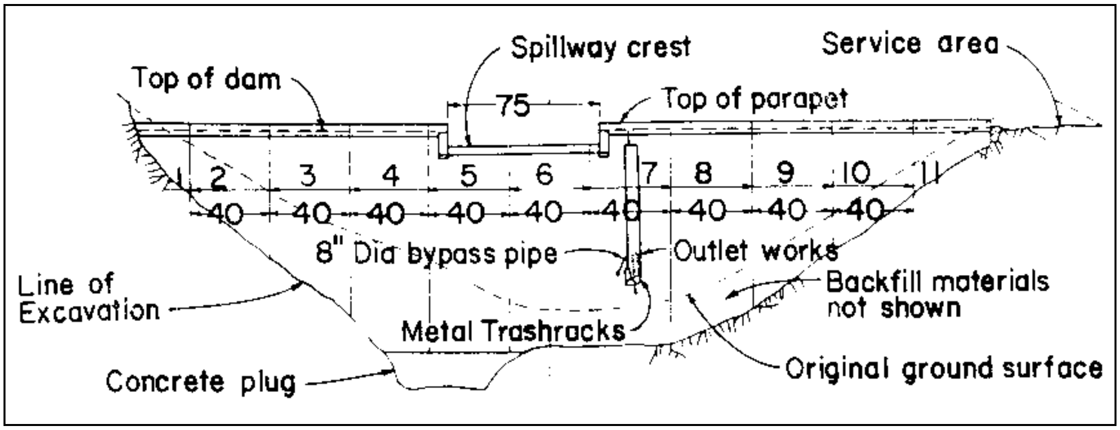

Figure 3-17. Upstream elevation of Wildhorse Dam. Note dental concrete (concrete plug) (USBR)

additional thickness along the abutment is called a socle in Portugal. Arch dams in nonsymmetrical sites can be designed more efficiently by constructing a pad along the abutment of the long side. Reduced excavation is accomplished by filling in a single but prominent depression with a pad rather than excavating the entire abutment to smooth the profile. The pad, because it is analogous to a spread footing, will reduce pressure and deformation, especially on weak rock.

(1) The pad in cross section has the geometric shape of a trapezoid. The top surface of the trapezoid then becomes the profile for the arch dam. To develop a smooth profile for the arch dam, the trapezoidal height will vary along the contact. The size of the footing is a function of the arch abutment thickness. At the contact of the arch and the pad, the pad thickness is greater than the arch thickness by a nominal amount of 5 feet, or greater, as determined from two-dimensional (2-D) finite element studies. The extra concrete thickness, analogous to a berm, is constructed on both faces. This berm provides geometric and structural delineation between the dam and the pad. Below the berm, the pad slopes according to the canyon profile. The slope is described in a vertical radial plane. In a wide valley where gravity action is predominant, a nominal slope of 1:1 is suggested on the downstream side. In narrow canyons, where arch action is the major structural resistance, the slope may be steeper. The upstream face may be sloped or vertical depending on the loading combinations. For example, reservoir drawdown during the summer coupled with high temperatures will cause upstream deflection and corresponding larger compressive stresses along the heel. Thus, to simulate the abutment the upstream face should be sloped upstream. Otherwise, a vertical or near vertical face will suffice. Normally, a constant slope will be sufficient, both upstream and downstream. A pad should be used to fill depressions in the profile that would otherwise cause overexcavation and to smooth the profile or to improve the site symmetry.

(2) The appropriate shape should be evaluated with horizontal and vertical sections at points on the berm. Classical shallow beam structural analyses are not applicable because the pad is not a shallow beam. For completeness, check the shape for stress and stability at several representative locations around the perimeter. Loads to be considered in structural design of the pad include moment, thrust, and shear from the arch dam, as well as the

reservoir pressure. The load on the massive footing for a cantilever should also include its own weight. Two-dimensional FEMs are ideally suited to shape and analyze the various loads and load combinations. Reshaping and reanalysis then can also be easily accomplished.

(3) Construction of the pad may occur before or during construction of the arch dam. For example, if the foundation rock is very hard and/or massive, higher-strength concrete can be placed months before the arch is constructed. In this way, the pad concrete has time to cure and perhaps more closely approximate the actual foundation conditions. Or, as with the socle where a slightly different geometric shape is required at the arch abutment than at the top of the footing, the more efficient method is to construct the footing monolithic with the arch dam, in blocks, and lifts. Special forming details are necessary at the berm; above and below the berm, normal slip forming is sufficient. Artificial cooling and contraction joint grouting is recommended to avoid radial crack propagation into the dam from future shrinkage and settlement. As with the arch dam, no reinforcing steel is necessary in the foundation shaping concrete. Longitudinal contraction joints should be avoided to prevent possible tangential crack propagation into the dam. If, during construction, a significant crack should appear in the foundation or dam concrete and continue to run through successive lifts, a proven remedy is to provide a mat of reinforcing steel on the next lift of the block with the crack but not necessarily across contraction joints.

c. Thrust Blocks. Thrust blocks are another type of restitution concrete. These components are constructed of mass concrete on foundation rock and form an extension of the arch dam crest. They are particularly useful in sites with steep side slopes extending about three-fourths the distance to the top and then rapidly flattening. In such a site, significantly additional water storage can be achieved by thrust blocks without a proportional increase in costs. For small and short extensions beyond the neat line, the cross-sectional shape can simply be a continuation of arch dam geometry as shown on Figure 3-18; thus, for some distance past the neat line, arch action will resist some of the applied load. Beyond that distance, cantilever action resists the water load and must be stable as with a gravity dam. The extension may be a straight tangent or curved as dictated by the topography, as shown in Figure 3-19. If curved, the applied load is distributed horizontally and vertically, in which case the section can be thinner than a straight gravity section. If straight, the tangent section will exhibit some horizontal beam action, but conservatively, none should be assumed unless artificial cooling and contraction joint grouting are utilized. For cases where the thrust block sections are shaped as gravity dams, the analysis approximates the thrust block stiffness by reducing the foundation modulus on those arches connected to the thrust blocks. A reasonable value for the crest abutments is 100 kips per square inch (ksi). The abutment/foundation modulus should be linearly interpolated at elevations between the crest and the first lower arch abutting on rock. The reliability of this assumption should be tested by performing parametric studies with several different assumed rock moduli, comparing arch and cantilever stresses on each face, and noting the stress differences in the lower half of the dam. In general, stress differences should be localized around the thrust blocks.

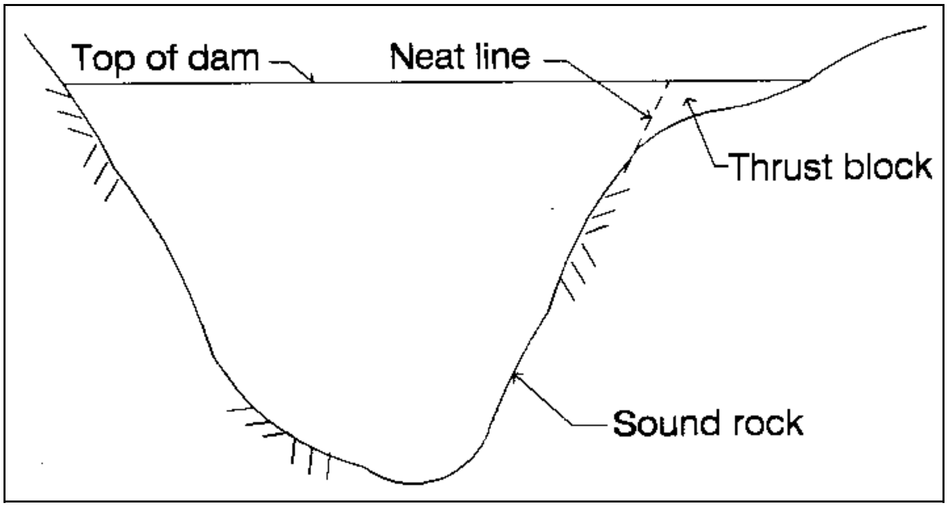

Figure 3-18. Schematic elevation of simple thrust block as a right abutment extension of an arch dam

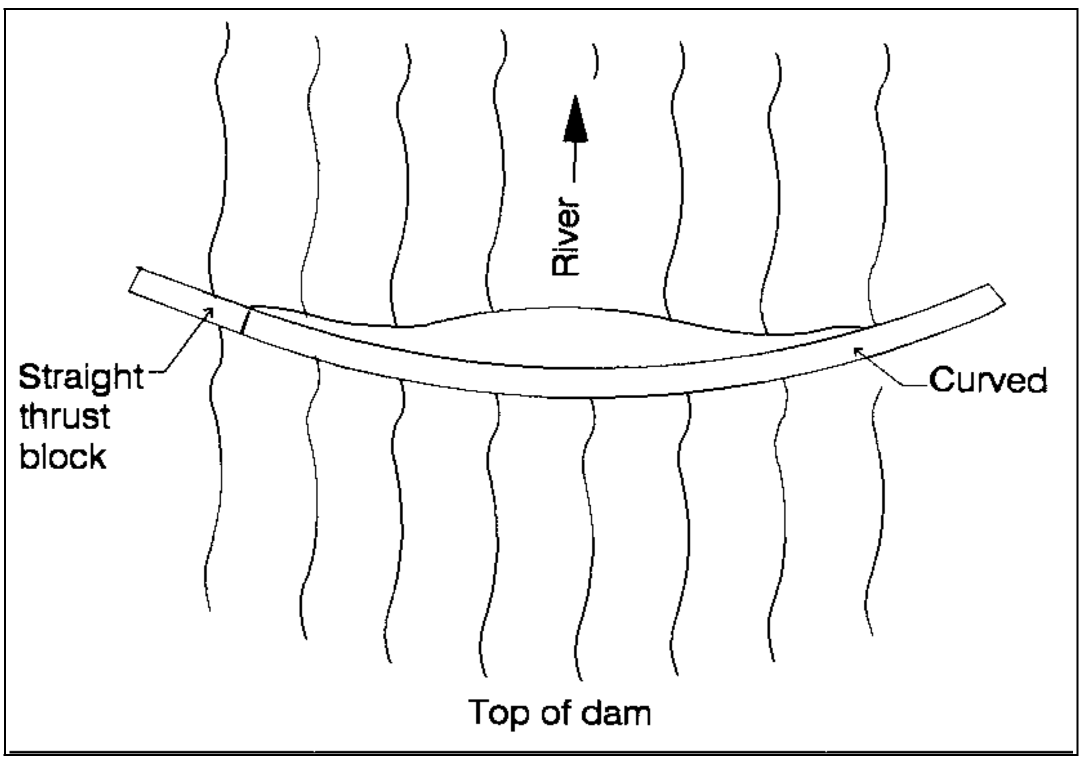

Figure 3-19. Schematic plan of straight and curved thrust blocks and water barrier

CHAPTER 4

LOADING COMBINATIONS

4-1. General. Arch dams are designed for the same loads as other dams with the exception of the temperature load which has a significant influence in arch dam design as compared to gravity dam design. The loads for which an arch dam must be designed are as follows:

a. Dead Load. Dead load is due to the weight of the concrete plus the appurtenant structures. The unit weight of the concrete is based on the laboratory test results of the mix design; however, for preliminary design a unit weight of 150 pounds per cubic foot (pcf) can be used. The weight of the appurtenances is normally negligible compared to the weight of the dam and may be neglected in static design. In the case of a massive, overflow-ogee-weir spillway or massive outlet works, it may be prudent to include these structures in the finite element model used for static and dynamic analyses.

b. Temperature Load. The temperature load results from the differences between the closure (grouting) temperature and concrete temperatures in the dam during its operation. The closure temperature is the concrete temperature at the time of grouting of the contraction joints. This temperature, which in effect is the datum for all the future temperature loading, is determined from the results of the stress analyses of the dam under different loading combinations. Another way to describe the closure temperature may be to consider it as a stress-free temperature (only for dams that are grouted). For example, if an arch dam is grouted at 55 °F, there will not be any stresses due to the temperature loading in the dam as long as the operating temperature of the dam remains at 55 °F. However, once the concrete temperature exceeds 55 °F, the resulting positive temperature loading will cause compressive stresses in the arches which in turn result in deflection into the reservoir. The opposite is true when in the winter the concrete temperature goes below 55 °F. In this case, the arches will experience tension which would cause deflection downstream. The selection of the closure temperature usually involves a compromise between the ideal stress distribution in the dam and practical considerations such as the feasibility of achieving the desired closure temperature. The closure (grouting) temperature is one of the most important construction parameters in arch dams because once the monolith joints are grouted, the structure is assumed to become monolithic and the arch action begins. Following the determination of the closure temperature, the individual blocks should be sized to prevent cracking during construction and to provide satisfactory contraction joint opening for grouting.

(1) The following hypothetical example may help explain the role of the temperature load - and that of the closure temperature - in arch dams. Consider an arch dam to be designed for a site with uniform air and water temperatures of 65 °F, i.e., no seasonal cyclic temperature changes in the air and reservoir. Neglecting the effect of the solar radiation, the operational concrete temperature is then the same as that of the air and reservoir: 65 °F. It is further assumed that there is no fluctuation in the reservoir level, so the dam is subjected to the full reservoir load at all times. Since the hydrostatic load in this example produces large tensile stresses along the heel of the cantilevers, the design objective would be to counteract the

tensile stresses by introducing a large temperature load which would cause the cantilevers to deflect into the reservoir. This objective can be accomplished by choosing the lowest possible closure temperature - say 35 ^{o}F - which would result in a 30-^{o}F (65 ^{o}F - 35 ^{o}F) temperature load.

(2) As seen in the simplified example, the closure temperature is a design parameter which, within certain constraints, can be selected to help achieve desirable stress distribution in arch dams; thus, it has an effect on the geometry, i.e., the vertical and horizontal curvatures of the dam. Figure 4-1 shows the relationship between the closure temperature and the operating concrete temperatures which comprise the temperature loading as used for the Portugues Dam, Ponce, Puerto Rico.

c. Hydrostatic Load. The reservoir load is based on a study of the reservoir operation. Unlike a gravity dam for which higher reservoir levels would result in more critical cases, an arch dam may experience higher tensile stresses (on the downstream face) under low reservoir elevations. Studies of the reservoir operation should include the frequency of occurrence and duration of reservoir stages and the time of the year in which different water stages occur. These data are used in conjunction with the appropriate temperature information as shown in Figure 4-2 (see Chapter 8 for temperature study).

d. Earthquake Load. For arch dams in earthquake zones, two levels of earthquakes should be used. These are the Operational Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE). OBE is defined as a ground motion having a 50 percent chance of exceedance in 100 years. The dam is expected to respond elastically under the OBE (assuming continuous monolithic action along the entire length of the dam). MDE is the maximum level of ground motion for which the arch dam should be analyzed, and it is usually equated to the maximum credible earthquake (MCE). MCE is defined as the largest reasonable possible earthquake that could occur along a recognized fault or within a particular seismic source. If dam failure poses no hazard to life, an MDE lower than MCE level of motion may be specified. Under the MDE, the dam is allowed to respond nonlinearly and incur significant damage, but without a catastrophic failure in terms of loss of life or economics. Close coordination should be maintained with HQUSACE (CECW-ED) during the selection process of earthquake ground motions for arch dams.

e. Miscellaneous Loads. Where applicable, loads due to ice and silt should be included in the design of an arch dam. In the absence of design data, an ice load of 5 kips per linear foot of contact along the axis may be assumed. The silt load should be determined from the results of the sedimentation study for the dam. If these loads are small compared to the other loads, they can be neglected at the discretion of the designer.

4-2. Loading Combinations. Arch dams are designed for two groups of loading combinations. The first group combines all the static loads and the second group takes into account the effects of earthquake. Depending on the probability of occurrence of the cases in each group, they are labeled as Usual, Unusual, and Extreme loading cases. It must be stressed that each dam is a unique structure, and there are many factors to consider when deciding on the loading combinations. Factors such as climatic conditions, purpose of reservoir, spillway usage, operation of reservoir (as designed and anticipated

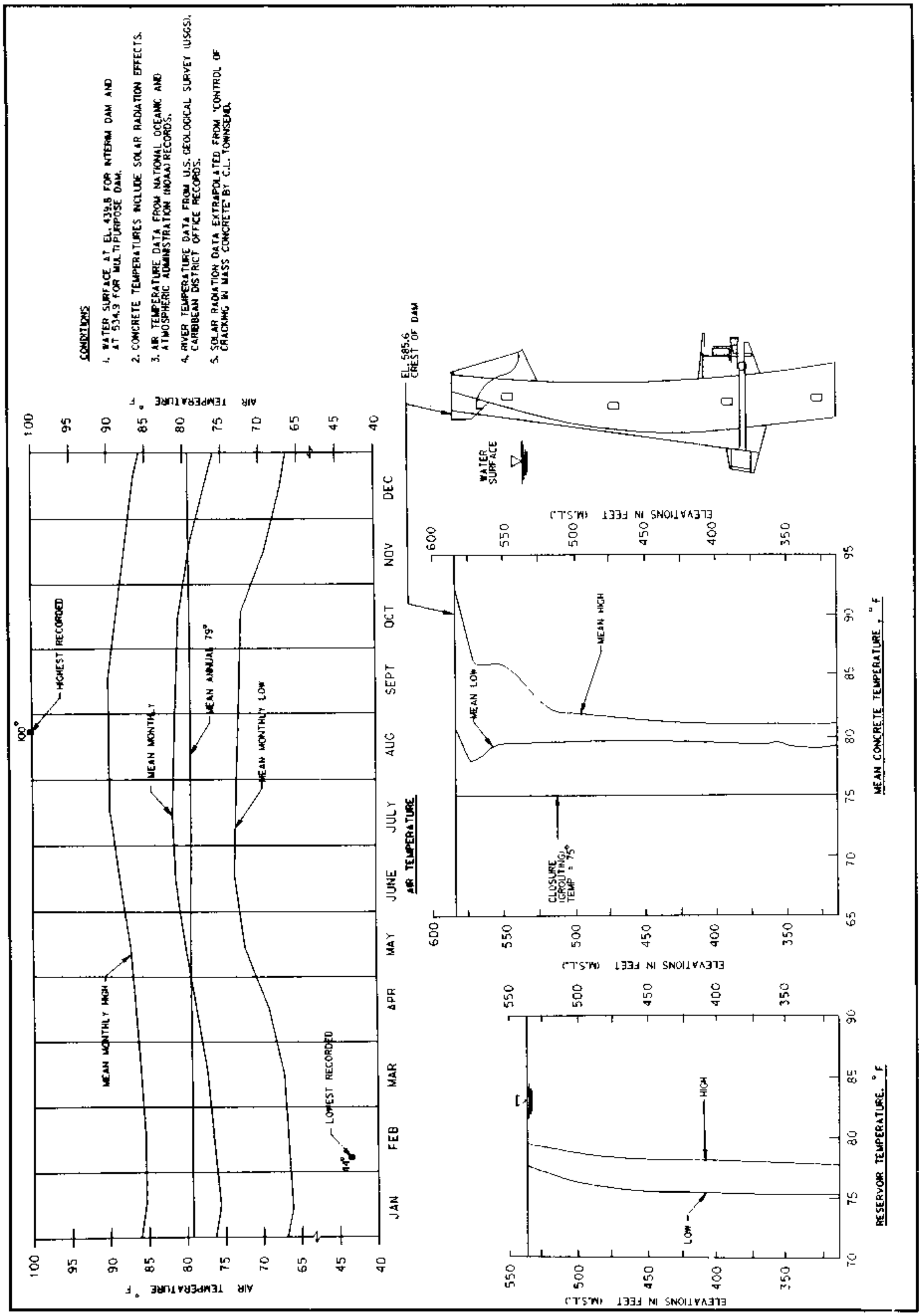

Figure 4-1. Mean concrete temperature

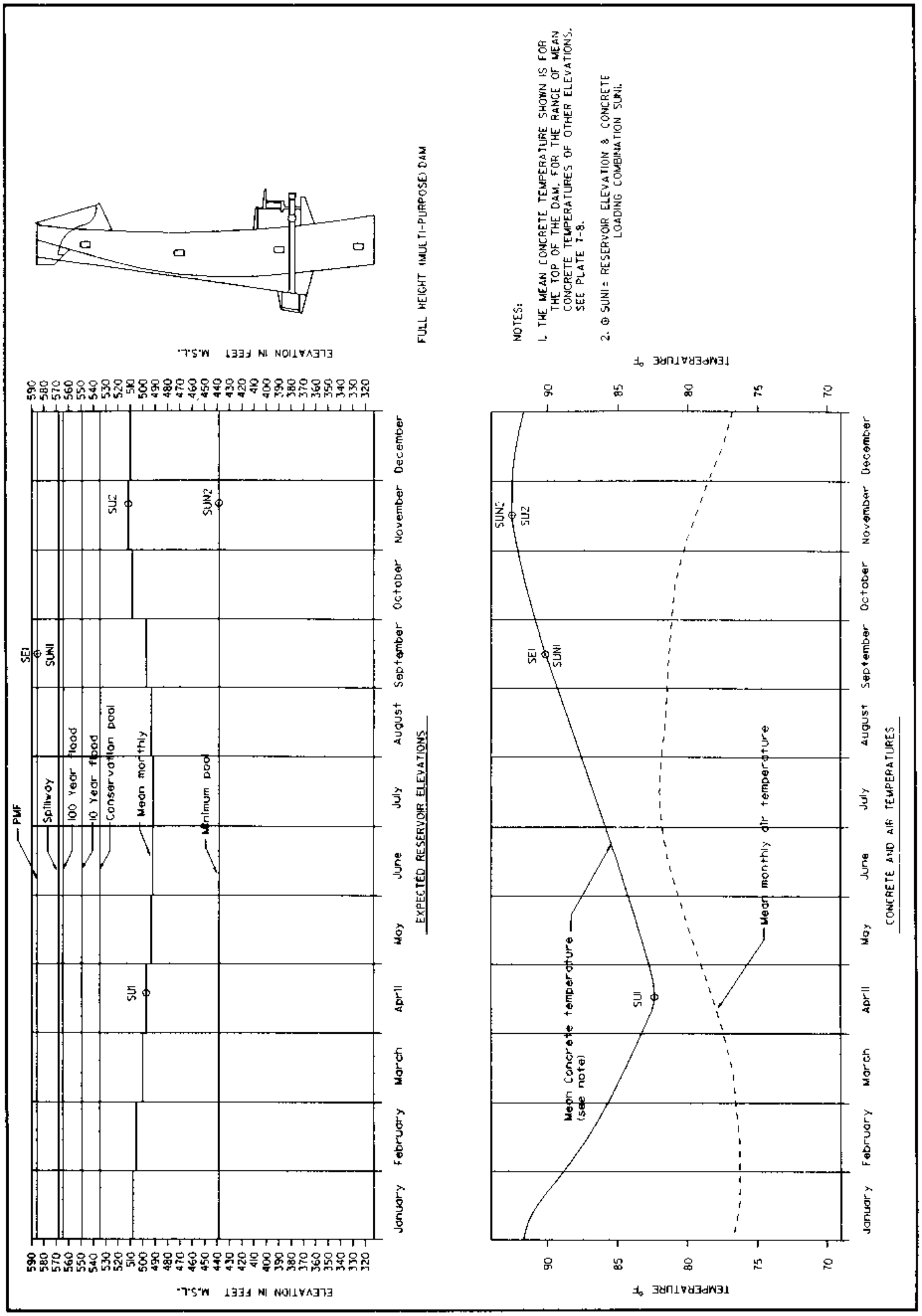

Figure 4-2. Reservoir elevations, mean concrete and air temperatures

actual), and location have direct bearing on the approach taken in determining and combining the loads and the classification of the loading combinations. Figure 4-2 illustrates the selection of the loading combinations for the Portugues Dam, Ponce, Puerto Rico. Tables 4-1 and 4-2 present the static and dynamic loading combinations which must be investigated as a minimum. The allowables and factors of safety are discussed in Chapter 11.

4-3. Selection of Load Cases for Various Phases of Design.

a. Reconnaissance. No detailed design is required during reconnaissance studies. The study in this phase is limited to the volume computation as discussed in Chapter 6 for the purpose of comparative studies with other types of dams. Of course, it is assumed that a suitable site exists for an arch dam based on the geometry of the site and the type of foundation material.

b. Feasibility. Limited design work should be accomplished during this phase of the design. From the results of the basic hydrology study, the preliminary loading combinations should be established and the temperature loading on the dam should be estimated from a study of similar projects. Basic dimensions of the dam should be determined using the procedure discussed in Chapter 6 to the extent necessary to obtain the data required for stress analysis. Only a static design analysis using two opposing loading combinations is required at this time. Based on the results of the stress analysis, a number of trials and adjustments in the geometry may be required to obtain acceptable stress distribution throughout the dam.

c. Preconstruction Engineering and Design (PED). Detailed design and analysis of the dam are to be performed during the PED phase with the results presented in various Feature Design Memoranda (FDM). The load cases must be established at the earliest stages of this phase. The temperature loading needs to be determined from the results of the temperature study which is initiated at beginning of PED, and the final reservoir elevations, their durations, and the time of year in which these reservoir stages are expected must be determined in order to develop the loading combinations as shown in Figure 4-2. The selection of the loads for the loading combinations should be given careful consideration. As an example related to Figure 4-2, it is considered prudent to select the middle of September for the probable maximum flood (PMF), although theoretically the PMF could happen at any time during the 12-month cycle. The rationale is that it is more likely for the PMF to occur in the middle of the wet season - for this particular site - than any other time. The significance of the above example is in the "concrete temperature occurring at that time," in accordance with case SE1, Table 4-1. If the PMF is assumed to happen in April when the mean concrete temperature is at its lowest, there would be a very small temperature on the dam (as shown by the closure temperature and the mean low concrete curve in Figure 4-1) which would result in too much conservatism. The opposite would be true if the PMF is assumed in November.

TABLE 4-1

Static Loading Combinations

Static Usual (SU)

SU1: Minimum usual concrete temperature.
Reservoir elevation occurring at that time. Dead Load.

SU2: Maximum usual concrete temperature.
Reservoir elevation occurring at that time. Dead Load.

SU3: Normal Operating Reservoir Condition.
Concrete temperature occurring at that time. Dead Load.

Static Unusual (SUN)

SUN1: Reservoir at spillway crest elevation.
Concrete temperature at that time. Dead Load.

SUN2: Minimum design reservoir elevation.
Concrete temperature occurring at that time. Dead Load.

SUN3: End of construction condition. Structure completed, empty reservoir. Temperature Load.

Static Extreme (SE)

SE1: Reservoir at Probable Maximum Flood (PMF) elevation. Concrete temperature occurring at that time. Dead Load.

TABLE 4-2

Dynamic Loading Combinations[1]

Dynamic Unusual (DUN)

DUN1: Operating Basis Earthquake (OBE) plus static load case SU3.

DUN2: OBE plus static load case SUN3.

Dynamic Extreme (DE)

DE1: Maximum Design Earthquake (MDE) plus static load case SU3.

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[1] See Chapter 11 for guidelines for treatment of dynamic response of the dam.

CHAPTER 5

DESIGN LAYOUT

5-1. General Design Process. Design of an arch dam involves the layout of a tentative shape for the structure, preliminary static stress analysis of this layout, evaluation of the stress results, and refinement of the arch dam shape. Several iterations through the design process are necessary to produce a satisfactory design which exhibits stress levels within an acceptable range. The final dam layout that evolves from the iterative design process is then statically analyzed by the finite element method. "Static analysis" refers to the analysis performed after a layout has been achieved through the design process. "Preliminary stress analysis" refers to the method of analysis performed during the iterative design process to investigate the state of stress for tentative layouts. The computer program ADSAS (Arch Dam Stress Analysis System) (USBR 1975) is used for the preliminary stress analysis and is discussed in more detail in paragraph 5-5. GDAP (Graphics-based Dam Analysis Program) (Ghanaat 1993a) is a special purpose finite element program, specifically developed for the analysis of arch dams. GDAP is discussed in more detail in Chapter 6. Preliminary stress analyses are relatively quick and inexpensive to run compared to the static analysis which is more detailed, both in its input as well as its output. Although past history has shown that results from both procedures are comparable, an arch dam layout that reaches the static analysis phase may still require refinement, pending evaluation of static analysis results.

5-2. Levels of Design. There are three phases of the life cycle process of a project for which layouts are developed; reconnaissance, feasibility, and PED. The degree of refinement for a layout is determined by the phase for which the design is developed.

a. Reconnaissance Phase. Of the three phases mentioned in paragraph 5-2, the least amount of engineering design effort will be expended in the reconnaissance phase. Examination of existing topographic maps in conjunction with site visits should result in the selection of several potential sites. When selecting sites during the reconnaissance phase, emphasis should be placed on site suitability, i.e., sites with adequate canyon profiles and foundation characteristics.

(1) Reconnaissance level layouts for different sites can be produced quickly from the empirical equations discussed in paragraph 5-4. Empirical equations to determine concrete quantities from these layouts are presented in paragraph 5-6. Alternatively, the structural engineer may elect to base reconnaissance level layouts on previous designs which are similar in height, shape of profile, loading configuration, and for which stresses are satisfactory. However, it should be pointed out that most arch dams which have been constructed to date are single-center. Because the technology exists today which simplifies the layout of more efficient, multicentered arch dams, most dams that will be designed in the future will be of the multicentered variety. Therefore, basing a tentative, reconnaissance level layout on an existing single-center arch dam may result in conservative estimates of concrete quantity.

(2) Topographic maps or quad sheets that cover an adequate reach of river provide sufficient engineering data for this phase. From this region, one or more potential sites are selected. The areas around these sites are enlarged to 1:50 or 1:100 scale drawings. These enlarged topographic sheets and the empirical formulas in paragraphs 5-4 and 5-6 will produce the geometry and concrete volume for a reconnaissance level layout.

b. Feasibility Phase. Designs during the feasibility phase are used in the selection of the final site location and as a basis for establishing the baseline cost estimate. Feasibility designs are made in greater detail than reconnaissance designs since a closer approximation to final design is required.

(1) As a result of the work performed during the reconnaissance phase, the structural engineer should now have available one or more potential sites to evaluate during the feasibility phase. Using the iterative layout process discussed in paragraph 5-4, tentative designs will be plotted, analyzed, evaluated, and refined for each potential site until a proposed layout evolves that provides the best balance between minimal concrete volume and minimum stress level. Load cases to be analyzed during the feasibility phase are discussed in Chapter 4. From the sites evaluated and their respective layouts, the structural engineer will select the most economical design, and this will be carried into the PED phase. A baseline cost estimate will be developed for the final layout.

(2) The iterative layout process requires a certain amount of topographic and subsurface information. However, these data should be obtained with the knowledge that funds for feasibility studies are limited. Aerial topographic surveys of potential sites are required as well as a few core borings to determine an approximation of the depth of overburden. Loading conditions, as discussed in Chapter 4, should be defined.

c. Preconstruction Engineering and Design. Design work during this phase is presented in the FDM which is also used to develop contract plans and specifications. The final design layout produced during the feasibility stage is subjected to further static and dynamic analysis during the PED phase. Any remaining load cases that have not been analyzed during the feasibility phase should be analyzed and evaluated at this time. This may require that any missing data (operating conditions, thermal loads, etc.) be finalized prior to the analysis. If results from all of the preliminary stress analysis load cases indicate the final layout is acceptable, design may proceed to the static FEM analysis. Otherwise, the layout requires refinement.

5-3. Procedure. A single-center, variable-thickness, arch dam is assumed for the purpose of discussion. The procedure for laying out other types of arch dams differs only in the way the arches are defined.

5-4. Manual Layout. Although the term "Layout of an arch dam" implies a single procedure, layout actually consists of an iterative, refining process involving several layouts, each successive one improving on the previous. The first of these layouts require the structural designer to assume some initial parameters which will define the shape of the arch dam. As stated in paragraph 5-2a(2), a 1:50- or 1:100-scale topographic map of a dam site is required before layout begins. If possible, the contours should represent topography of foundation rock; however, in most instances, only surface topography is available at this stage of design. The structural designer must then assume a reasonable amount of overburden, based on core borings or sound judgement, to produce a topo sheet that reflects the excavated foundation.

a. Axis. The crest elevation required for the dam should be known at this time from hydrologic data and this, in conjunction with the elevation of the streambed (or assumed foundation elevation) at the general location of the dam, determines the dam height, H (feet). The structural designer should select a value for the radius of the dam axis (R_{AXIS}). For the initial layout where the engineer would have no reasonable estimate for the value of R_{AXIS} from a previous layout, the following empirical relationship has been derived by the USBR (Boggs 1977) based on historical data from existing dams:

$$R_{AXIS} = 0.6\ L_1 \tag{5-1}$$

where L_1 represents the straight line distance (in feet) measured (from the topo sheet) between abutments excavated to assumed foundation rock at the crest elevation. At this time, the structural designer should also measure the straight line distance between abutments excavated to assumed foundation rock at an elevation (el)[1] 0.15H above the base (L_2). See example in Figure 5-1. These three variables (H, L_1, and L_2) are also used to define an initial shape for the crown cantilever (paragraph 5-4c).

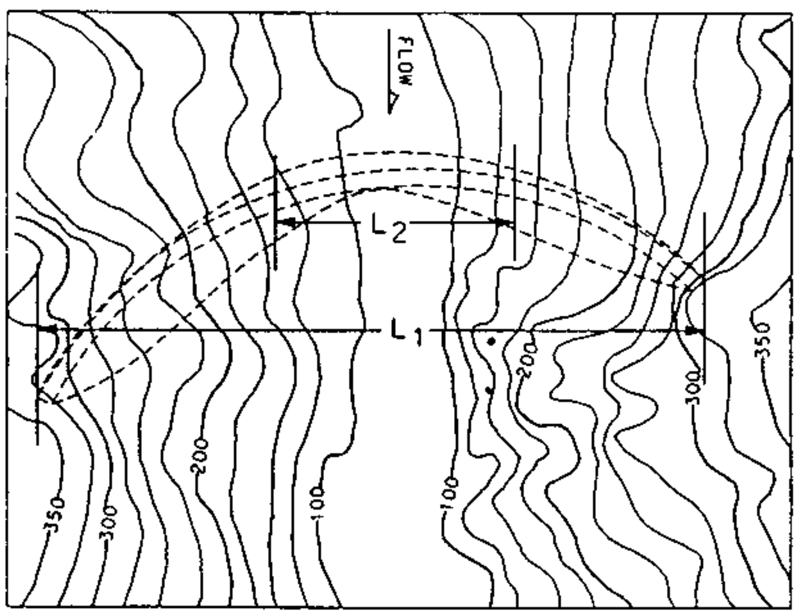


Figure 5-1. Determination of empirical values L_1 and L_2

[1] All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

(1) On a sheet of vellum or transparent paper, an arc is drawn with a radius equal to R_{AXIS} at the same scale as the topo sheet. This arc represents the axis of the dam. The vellum is then overlaid and positioned on the topo sheet so as to produce an optimum position and location for the dam crest; for this position, the angle of incidence to the topo contour at the crest elevation (β in Figure 2-1) should be approximately equal on each side. As shown in Figure 5-2, R_{AXIS} may require lengthening if the arc fails to make contact with the abutments or if the central angle exceeds 120 degrees.

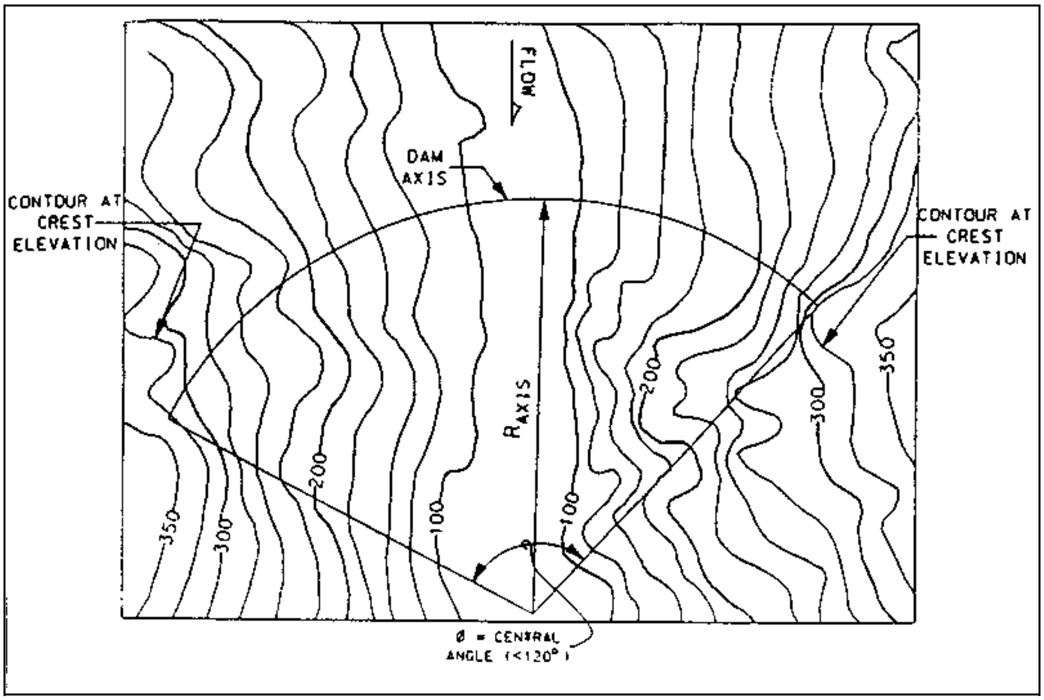


Figure 5-2. Layout of dam axis

(2) The magnitude of the central angle of the top arch is a controlling value which influences the curvature of the entire dam. Objectionable tensile stresses will develop in arches of insufficient curvature; such a condition often occurs in the lower elevations of a dam having a V-shape profile. The largest central angle practicable should be used considering that the foundation rock topography may be inaccurately mapped and that the arch abutments may need to be extended to somewhat deeper excavation than originally planned. Due to limitations imposed by topographic conditions and foundation requirements, for most layouts, the largest practicable central angle for the top arch varies between 100 and 120 degrees.

b. <u>Location of Crown Cantilever and Reference Plane.</u> On the overlay, locate the crown cantilever at the intersection of the dam axis and the lowest point of the site topography (i.e., the riverbed). This corresponds to the point of maximum depth of the dam. A vertical plane passing through this point and the axis center represents the reference plane (or plane of centers). On the overlay plan, this plane is shown as a line connecting the crown cantilever and the axis center. Later, when arcs representing arches at other elevations are drawn, they will be located so that the centers of the arcs will be located on the reference plane. Ideally, the reference plane should be at the midpoint of the axis. This seldom occurs, however, because most canyons are not symmetrical about their lowest point.

c. Crown Cantilever Geometry. The geometry of the crown cantilever controls the shape of the entire dam and, as a result, the distribution and magnitude of stresses within the body. The empirical equations which follow can be used to define thicknesses of the crown at three locations; the crest, the base, and at el 0.45H above the base:

$$T_C = 0.01(H + 1.2L_1) \tag{5-2}$$

$$T_B = \sqrt[3]{0.0012HL_1L_2\left(\frac{H}{400}\right)^{\frac{H}{400}}} \tag{5-3}$$

$$T_{0.45} = 0.95T_B \tag{5-4}$$

(1) In addition, upstream and downstream projections of the extrados (upstream) and intrados (downstream) faces can also be arrived at empirically. Those relationships are:

$$USP_{CREST} = 0.0 \tag{5-5}$$

$$USP_{BASE} = 0.67T_B \tag{5-6}$$

$$USP_{0.45H} = 0.95T_B \tag{5-7}$$

$$DSP_{CREST} = T_C \tag{5-8}$$

$$DSP_{BASE} = 0.33T_B \tag{5-9}$$

$$DSP_{0.45H} = 0.0 \tag{5-10}$$

(Note: These empirical equations were developed by the USBR and are based on historical data compiled from existing dams. However, the engineer is not restricted to using the parameters derived from the empirical equations; they are presented as an aid for developing initial parameters and only for the first layout. Sound engineering judgement resulting from experience obtained in arch dam layout may also be utilized when defining an initial layout or refining a previous one. Values for subsequent layouts will consist of adjustments, usually by engineering evaluation of stress analysis, of the values used in the previous iterations.)

(2) As shown in Figure 5-3, the upstream and downstream projections at the crest, base, and at el 0.45H above the base can now be plotted in elevation in reference to the dam axis. This plot is referred to as the "plane of centers" view. The next step is to define the upstream and downstream faces of the crown cantilever using a circular arc (or combinations of straight lines and circular arcs) which passes through the upstream and downstream projection points as shown in Figure 5-4. With the faces defined in this

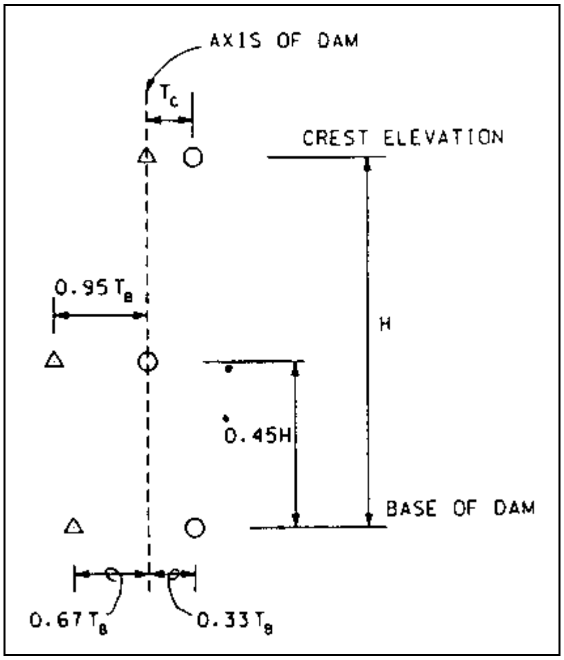


Figure 5-3. Empirically derived projections of the crown cantilever

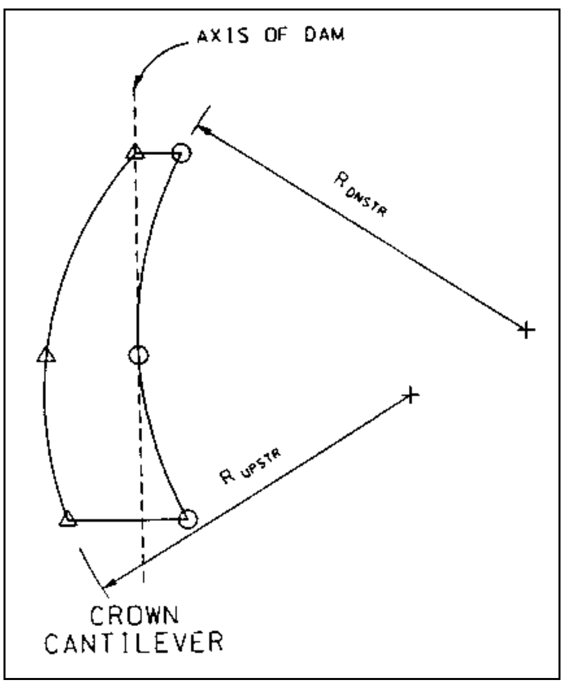


Figure 5-4. Definition of upstream and downstream faces

manner, upstream and downstream projections at any elevation can be obtained. This information will be necessary when laying out the arches.

d. Estimating the Dam Footprint. The axis of the dam on the topographic overlay corresponds to the upstream face of the dam at the crest. An arc representing the downstream face of the crest can be drawn with the center of the arc at the axis center and a radius equal to R_{AXIS} reduced by the thickness at the crest, T_C. On the plan overlay, three points are identified to aid in laying out the contact line between the foundation and the upstream face of the dam. Two of the points are the intersection of the axis of the dam with the foundation contour at the crest elevation at each abutment (points A and B). The third point is the upstream projection of the crown cantilever at the base. This point can be plotted in reference to the axis of the dam based on information taken from the plane of centers view (Figure 5-5). Using a french curve, a smooth curve is placed beginning at the upstream face of the crest on one abutment, passing through the upstream projection of the crown cantilever at the base (point C), and terminating at the upstream face of the crest at the other abutment (points A and B in Figure 5-6).

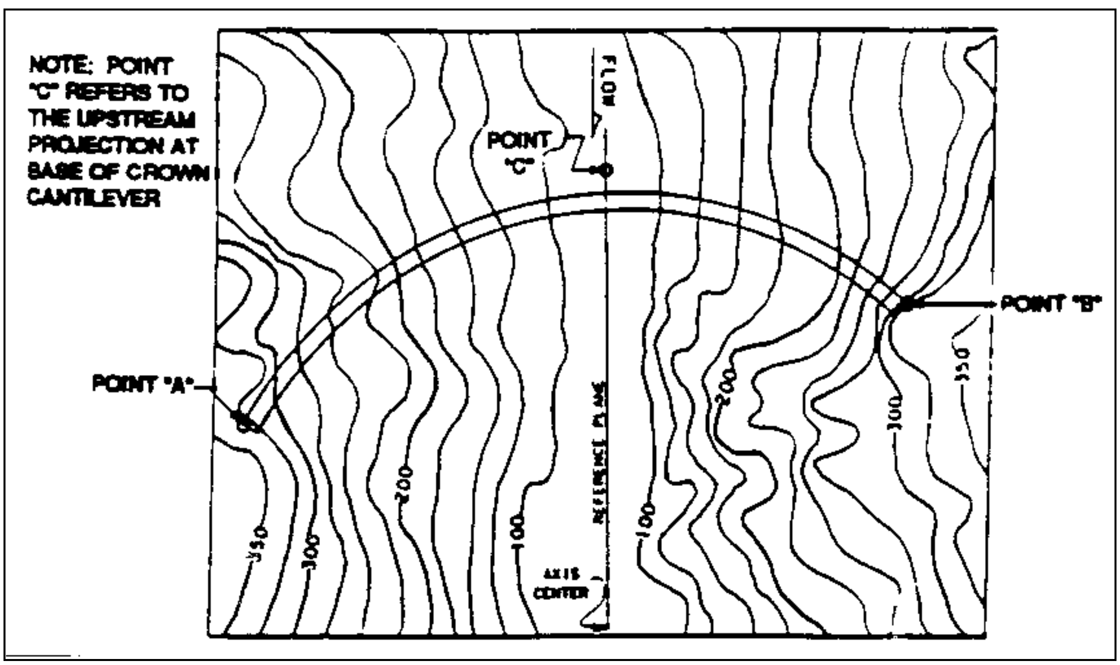


Figure 5-5. Contact points between dam and foundation at crest and crown cantilever

e. Layout of the Arches. Of all that is involved in arch dam layout, this step is possibly the most difficult. For shaping and analysis purposes, between 5 and 10 evenly spaced horizontal arches are drawn. These arches should be spaced not less than 20 feet nor greater than 100 feet apart. The lowest arch should be 0.15H to 0.20H above the base of the crown cantilever.

(1) Beginning at the arch immediately below the crest, determine, from the plane of the centers view, the upstream and downstream projections of the crown cantilever at that specific arch elevation. These projections are then

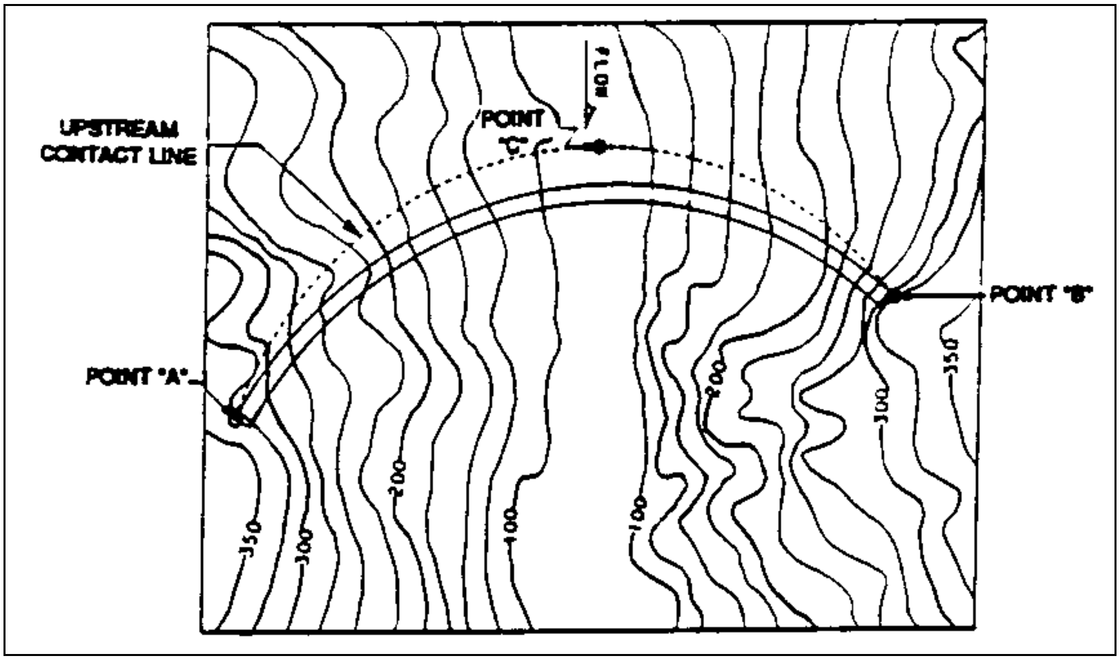


Figure 5-6. Upstream contact line at dam-foundation interface

plotted on the plan view along the reference plane. Using a beam compass, trial arcs representing the upstream face of the dam at that specific arch elevation are tested until one is found which meets the following criteria:

(a) The arc center <u>must</u> lie along the reference plane.

(b) The arc <u>must</u> pass through the upstream projection of the crown cantilever as plotted on the plan view.

(c) Both ends of the arc <u>must</u> terminate on the upstream contact line at a foundation elevation equal to or slightly deeper than the arch elevation.

(2) Locating an arch which satisfies all of these criteria is a trial and error process which may not be possible with a single-center layout. This is generally the case when dealing with unsymmetrical canyons where different lines of centers are required for each abutment (Figure 1-4). Figure 5-7 shows an example of an arch that meets the criteria. Of particular importance is that the ends of the arch must extend <u>into</u> the abutments and not fall short of them. This ensures that a "gap" does not exist between the dam and foundation.

(3) This procedure is repeated to produce the downstream face of the arch. Similar to what was performed for the upstream face, the downstream projection of the crown cantilever is determined from the plane of centers view and plotted on the plan view. The beam compass is then used to locate an arc that meets the three criteria with the exception that the arc must pass through the downstream projection of the crown cantilever with ends that terminate on the radial to the extrados at the abutment (Figure 5-7). If the same arch center is used for the upstream and downstream faces, a uniform thickness arch is produced. If the arch centers do not coincide, the

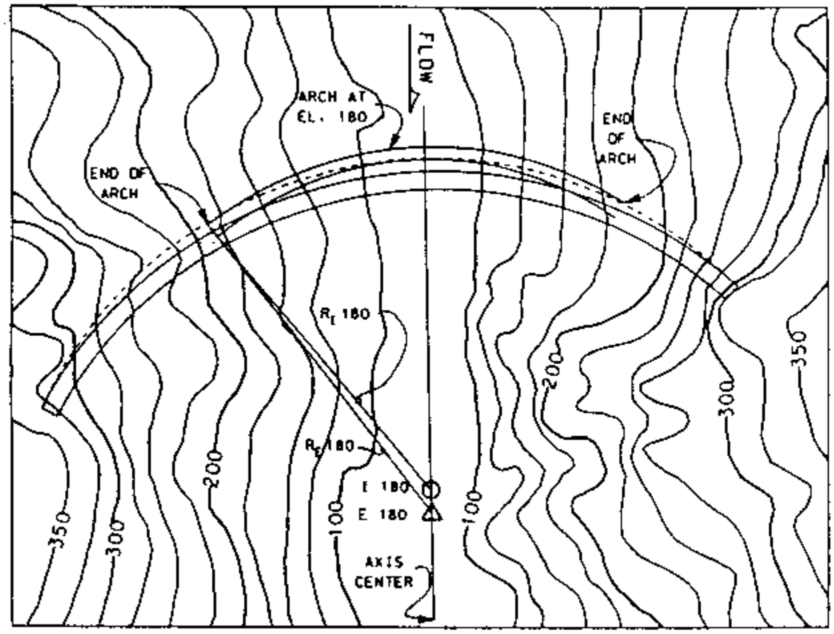


Figure 5-7. Typical layout of an arch section

arch produced will vary in thickness along its length (variable thickness arch).

(4) Once a satisfactory arch has been produced, the locations of the arch centers for the upstream and downstream face are marked along the reference plane on the plan view. Standard practice is to identify the extrados (upstream) face with a triangle (△) and the intrados (downstream) face with a circle (○). The corresponding arch elevation should be identified with each. See Figure 1-5 for an example. Although these arch centers appear to lie on a straigh line in plan, they all are positioned at their respective arch elevations, and it is highly unlikely that they are on a straight line in three-dimensional (3-D) space.

(5) Arches at the remaining elevations are plotted in similar fashion. Of particular importance to producing an acceptable plan view is to ensure that the footprint, when viewed in plan, is smooth and free flowing with no abrupt changes or reverse curvature. This requirement is usually met by the fact that a footprint is predetermined; however, endpoints of arches may not terminate exactly on the footprint. Revision of the footprint is necessary to ensure that it passes through all actual arch endpoints prior to checking it for smoothness.

f. Reviewing the Layout. Layout of an arch dam includes the preparation of three different drawings. The first is the plan view, which begins with locating a crest and ends with plotting the arches. The second drawing is a section, in elevation, along the reference plane, called the plane-of-centers view. This view has been partially produced when the crown cantilever was created, but it requires expansion to include the lines of centers, as will to be discussed in paragraph 5-4f(1). The third drawing to be produced is a profile (looking downstream) of the axis of the dam and the foundation.

Proper review includes examining all views for "smoothness," because abrupt changes in geometry will result in excessive stress concentrations. The term "smoothness" will be discussed in the following paragraph. Only when the plan view, plane of centers, and profile demonstrate "smoothness" and are in agreement is the layout ready for preliminary stress analysis. It should be pointed out that all three views are dependent on each other; when making adjustments to the geometry, it is impossible to change parameters in any view without impacting the others.

(1) Creating and Reviewing the Plane-of-centers View. In addition to the crown cantilever, the plane of centers also includes the lines of centers for the upstream and downstream face. A section is passed along the reference plane in plan to produce the plane-of-centers view. Each arch center, upstream and downstream, is plotted in elevation in reference to the axis center, as shown in Figure 5-8. The lines of centers are produced by attempting to pass a smooth curve through each set of arch centers (Figure 5-9). These lines of center define the centers for all arches at any elevation. If the curve does not pass through the arch centers located during the arch layout procedure, those arch centers will be repositioned to fall on the appropriate line of centers. Those particular arches will require adjustment on the plan view to reflect the change in position of the arch center. The structural engineer should understand that this adjustment will involve either lengthening or shortening the radius for that particular arch which will impact where the ends of the arch terminate on the abutments. Lines of centers should be smooth flowing without abrupt changes and capable of being emulated using combinations of circular curve and straight line segments, as shown in the example on Figure 5-10. The circular arcs and straight line segments used to define the lines of centers will be input into ADSAS when performing the preliminary stress analysis.

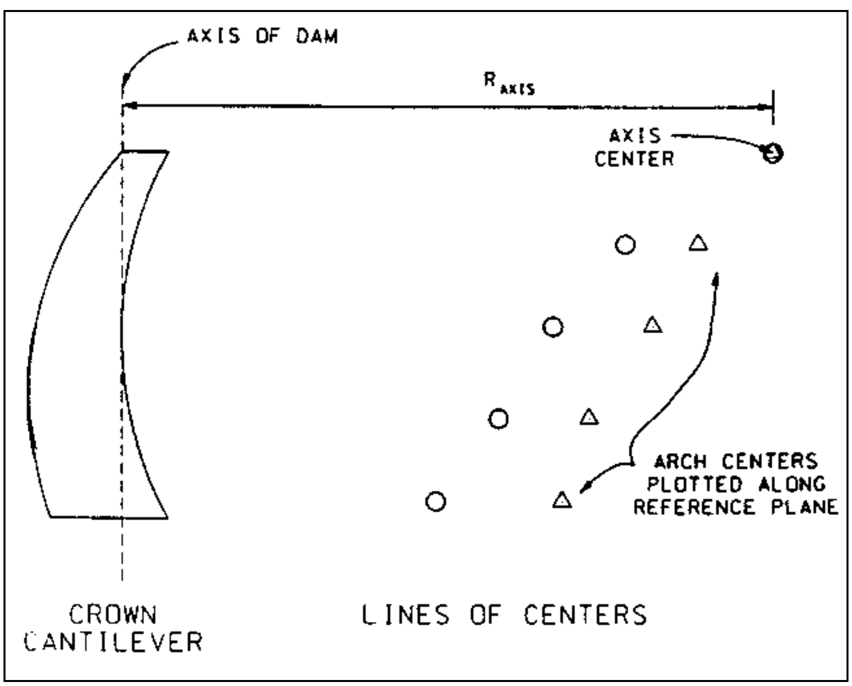


Figure 5-8. Plotting of arch centers along reference

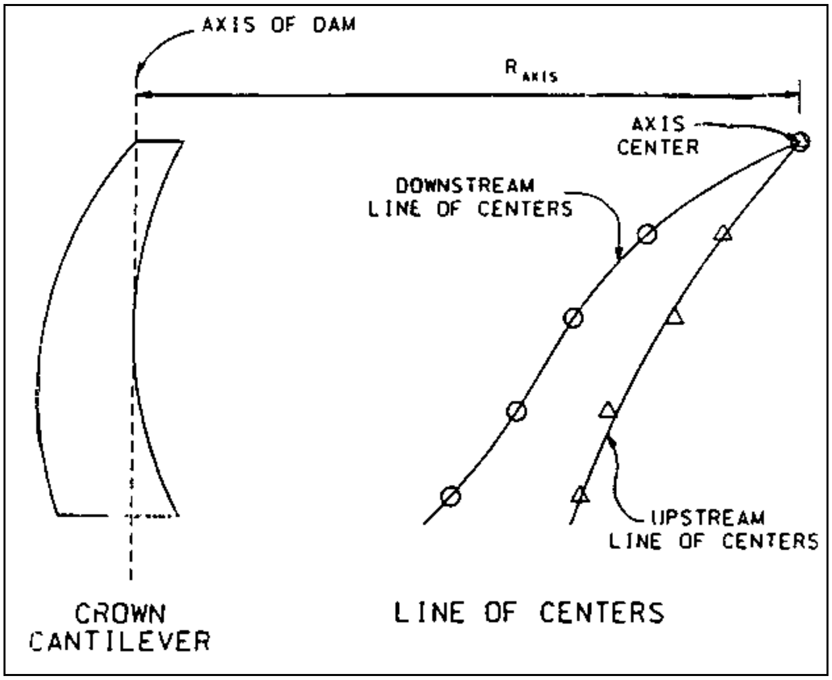


Figure 5-9. Development of the lines of centers

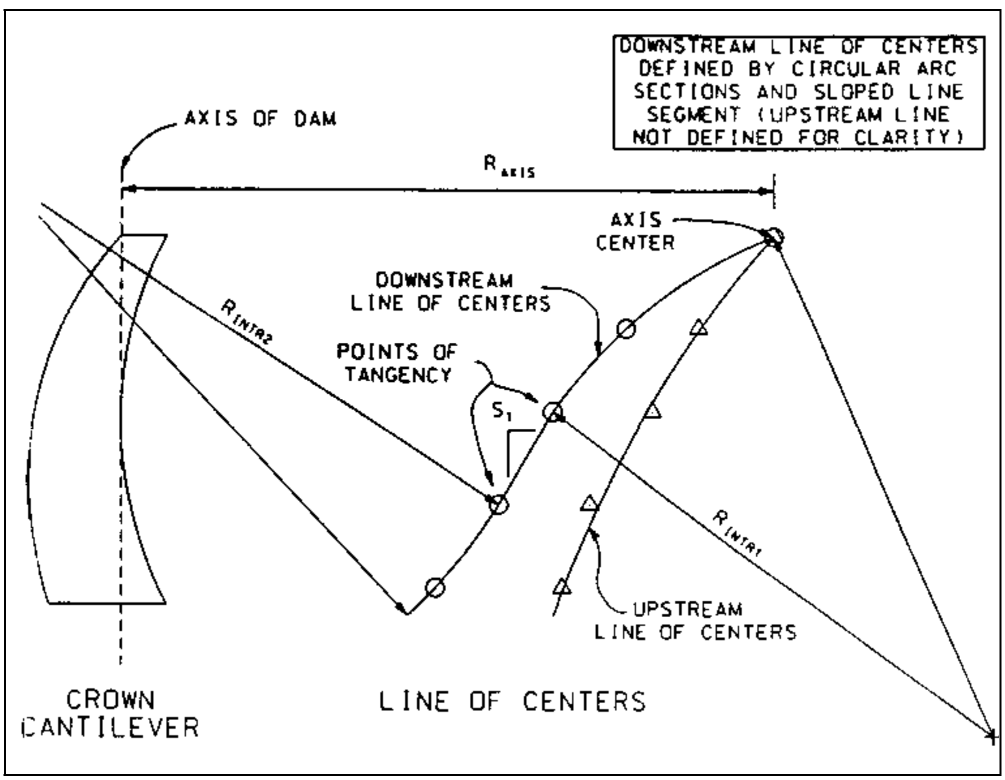


Figure 5-10. Defining the lines of centers

(2) Developing the Profile View. Once a satisfactory plan view and plane-of-centers view has been obtained, the profile view is ready to be created. The profile view is used to examine the amount of excavation that a particular layout has induced. The profile view consists of a developed elevation of the upstream face of the dam (looking downstream) with the foundation topography shown. It should be noted that this is a developed view rather than projection of the upstream face onto a flat plane. This "unwrapping" results in a view in which no distortion of the abutments exists. Figures 5-11 and 5-12 show, respectively, examples of acceptable and unacceptable profiles.

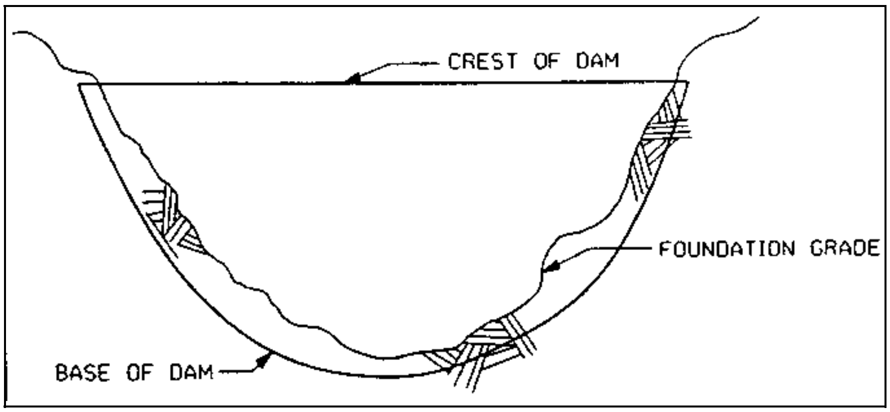


Figure 5-11. Example of an acceptable developed profile view

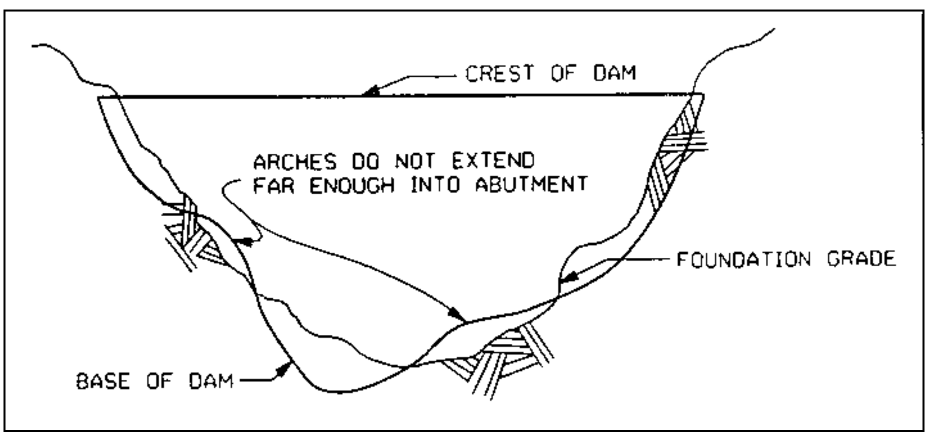


Figure 5-12. Example of an unacceptable developed profile view

(3) Foundation of Dam. In general, the foundation of the dam should be as the lines of centers: smooth and free flowing with no abrupt changes in geometry. The base of the dam must also extend into the foundation; otherwise, an undesirable condition develops in that "gaps" will occur between the base of the dam and the foundation, requiring foundation treatment. Pronounced anomalies should be removed by reshaping the affected arches until a smooth profile is obtained.

5-5. Preliminary Stress Analyses. After a satisfactory layout has been obtained, a preliminary stress analysis is performed to determine the state of stress of the dam under various loading conditions. The computer program ADSAS was developed by the USBR (1975) for this purpose. The Corps of Engineers adapted ADSAS for use on a microcomputer. ADSAS is based on the trial load method of analysis. A discussion on the theory of the trial load method is beyond the scope of this document; however, the USBR (1977) addresses this topic in detail.

a. ADSAS Input. While an exact description of the steps necessary to prepare an input data file for ADSAS is documented in the operations manual (USBR 1975) and an upcoming Corps of Engineers' manual. A brief description is included here. A typical ADSAS input data file contains four groups of information: a geometry definition section, material properties, loading conditions, and output control cards.

(1) Geometry Definition. Critical to the success of obtaining an accurate analysis is the ability to convey to the program the geometry which defines the shape of the arch dam. This geometry consists of:

(a) Crown cantilever geometry. Base elevation, crest elevation, projections of the upstream and downstream faces at the crest and the base, X and Y coordinates, and radii of all circular arcs used in defining the upstream and downstream faces, and slopes of any straight line segments used in defining the upstream and downstream faces.

(b) Lines-of-centers geometry. Axis radius, X and Y coordinates and radii of any circular arcs used in defining all lines of centers, slopes of any straight line segments used in defining all lines of centers, elevations at intersections between segments defining lines of centers, and horizontal distances from axis center to intrados and extrados lines of centers.

(c) Arch geometry. Elevations of all arches, angles to abutments for all arches, and angles of compound curvature.

All data required for the crown cantilever and lines of centers geometry are taken from the planes-of-centers view while data required for the arch geometry should be available from the plan view.

(2) Materials Properties. ADSAS analysis also requires material properties of both the concrete and the foundation rock. These data include modulus of elasticity of the concrete and foundation rock, Poisson's ratio for the concrete and foundation rock, coefficient of thermal expansion of the concrete, and unit weight of concrete.

(3) Loading Conditions. During layout, only static loading conditions are analyzed. Static load cases are discussed in Chapter 4. ADSAS is capable of analyzing hydrostatic, thermal, silt, ice, tailwater loads, and dead weight.

(4) Output Control. ADSAS provides the user with the ability to toggle on or off different portions of the output to control the length of the report while capturing pertinent information.

5-6. Evaluation of Results. Evaluation requires a thorough examination of all the analytical output. Types of information to be reviewed are the crown cantilever description, intrados and extrados lines of centers, geometrical statistics, dead load stresses and stability of blocks during construction, radial and tangential deflections and angular deformations, loading distributions, arch and cantilever stresses, and principal stresses. If any aspect of the design is either incorrect or does not comply with established criteria, modifications must be made to improve the design.

a. Resultant Components. Evaluation of the arch dam may also include examination of the resultants along the abutments. These resultants are separated into three components; radial, tangential, and vertical. The combined radial and tangential resultant should be directed into the abutment rock. In the lower arches, that abutment may tend to parallel the surface contours or daylight into the canyon. Prudent engineering suggests that the resultant be turned into the abutment. The solution may be a combination of increasing stiffness in the upper arches or flexibility in the lower arches. The effect is mitigated by including the vertical component which then directs the total resultant downward into the foundation.

b. Volume of Concrete. One major factor of a layout that requires evaluation is the volume of concrete that is generated. ADSAS computes this volume as part of its output. If a quantity is desired without proceeding through a preliminary stress analysis (as for a reconnaissance layout), that value can be arrived at empirically by the following equation (see paragraph 5-4a for definition of variables):

$$v = 0.000002H^2L_2\frac{(H + 0.8L_2)}{L_1 - L_2} + 0.0004HL_1[H + L_1] \qquad (5\text{-}11)$$

The volume of concrete calculated in ADSAS or derived from Equation 5-11 does not reflect mass concrete in thrust blocks, flip buckets, spillways, or other appurtenances.

5-7. Improvement of Design. The best of alternative designs will have stresses distributed as uniformly as possible within allowable limits combined with a minimum of concrete. Where to terminate a design and accept a final layout based on these criteria are difficult in some dams with widely varying loading conditions, such as with a flood control dam which has periods of low and high reservoir elevations. The primary means of effecting changes in the behavior of the dam is by adjusting the shape of the structure. Whenever the overall stress level in the structure is far below the allowable limits, concrete volume can be reduced, thereby utilizing the remaining concrete more efficiently and improving the economy. Following are some examples of how a design can be improved by shaping.

a. Loads and Deflections. Load distribution and deflection patterns should vary smoothly from point to point. Often when an irregular pattern occurs, it is necessary to cause load to be shifted from the vertical cantilever units to the horizontal arches. Such a transfer can be produced by changing the stiffness of the cantilever relative to the arch.

b. Reshaping Arches. If an arch exhibits tensile stress on the downstream face at the crown, one alternative would be to reduce the arch thickness by cutting concrete from the downstream face at the crown while maintaining the same intrados contact at the abutment. Another possibility would be to stiffen the crown area of the arch by increasing the horizontal curvature which increases the rise of the arch.

c. Reshaping Cantilevers. When cantilevers are too severely undercut, they are unstable and tend to overturn upstream during construction. The cantilevers must then be shaped to redistribute the dead weight such that the sections are stable. Severe overhang will cause tension to develop on the upper upstream face, contraction joints in the affected area to close, and prevent satisfactory grouting.

d. Force-stress Relationship. Shaping is the key to producing a complete and balanced arch dam design. The task of the designer is to determine where and to what degree the shape should be adjusted. Figure 5-13 should be used to determine the appropriate changes to be made to the structural shape. If an unsatisfactory stress condition is noticed, the forces causing those stresses and the direction in which they act can be determined by Figure 5-13. For example, the equations of stress indicate which forces combine to produce a particular stress. Knowing the force involved and its algebraic sign, it is possible to determine its direction from the sign convention shown on the figure. With that information the proper adjustment in the shape can be made so that the forces act to produce the desired stresses.

5-8. Presentation of Design Layout. Figures and plates that clearly show the results of the design layout and preliminary stress analyses should be included in the FDM. Plates that illustrate and describe the detailed geometry of the arch dam include:

a. Plan View. Arches, arch centers, angles to the abutments, axis center, dam-foundation contact line, and dam orientation angle are some items that are included in this view of the dam overlaid on the site topography (Figure 1-5).

b. Section along Reference Plane. This plate includes all the information that defines the vertical curvature of the crown cantilever and the line(s) of centers (Figure 1-6).

c. Cantilever Sections. All cantilevers generated during the preliminary stress analysis should be shown. Showing the thickness at the base and at the crest of each cantilever is also recommended as shown in Figure 5-14.

d. Arch Sections. Arch sections generated as a result of the preliminary stress analysis should be plotted. Appropriate thicknesses at the reference plane and at each abutment should be shown for each arch as shown in Figure 5-15.

e. Profile. A profile, developed along the axis of the dam, should be presented, showing locations of cantilevers and the existing foundation grade as shown in Figure 5-16.

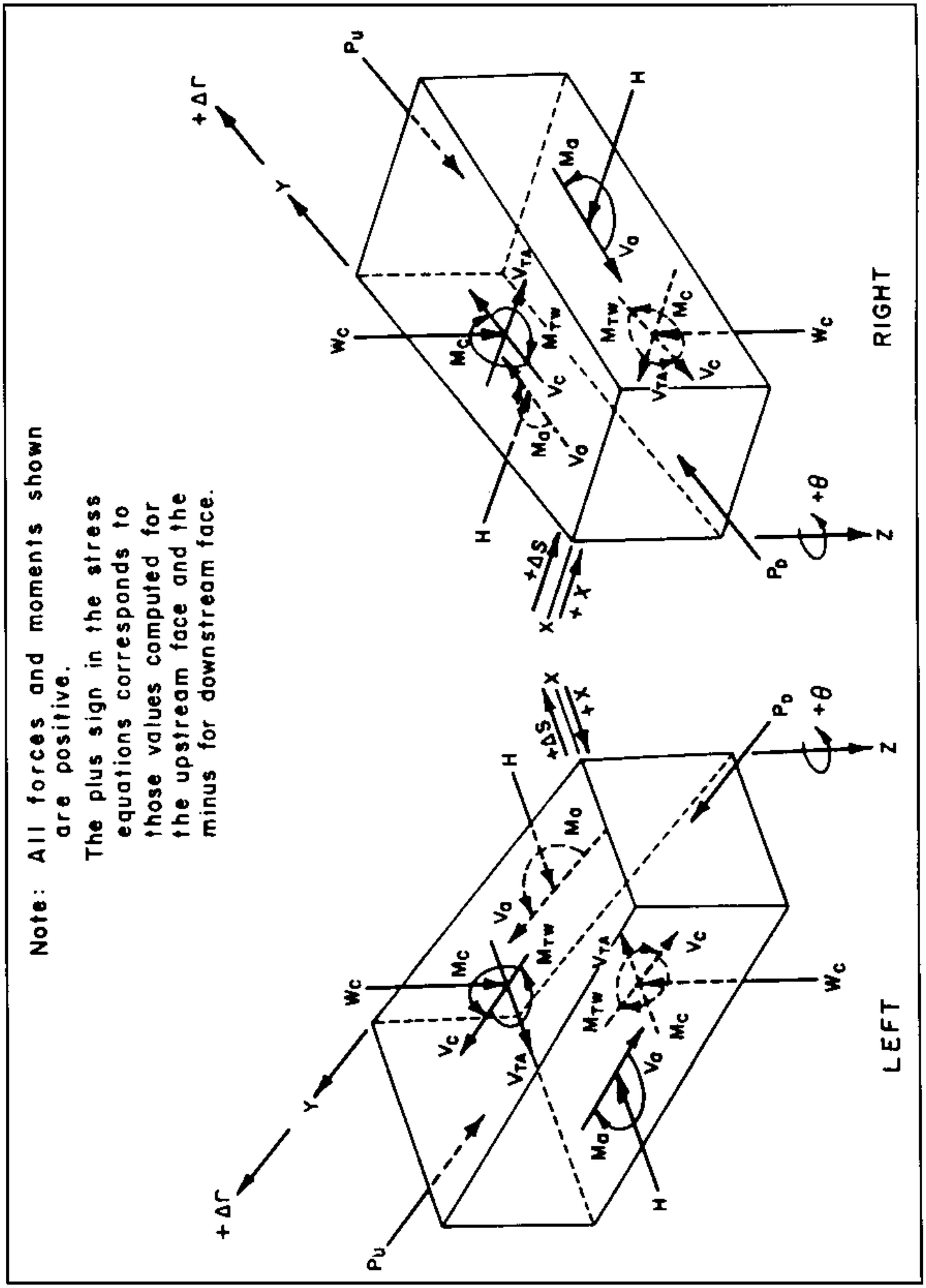


Figure 5-13. Sign convention for arch computations in ADSAS

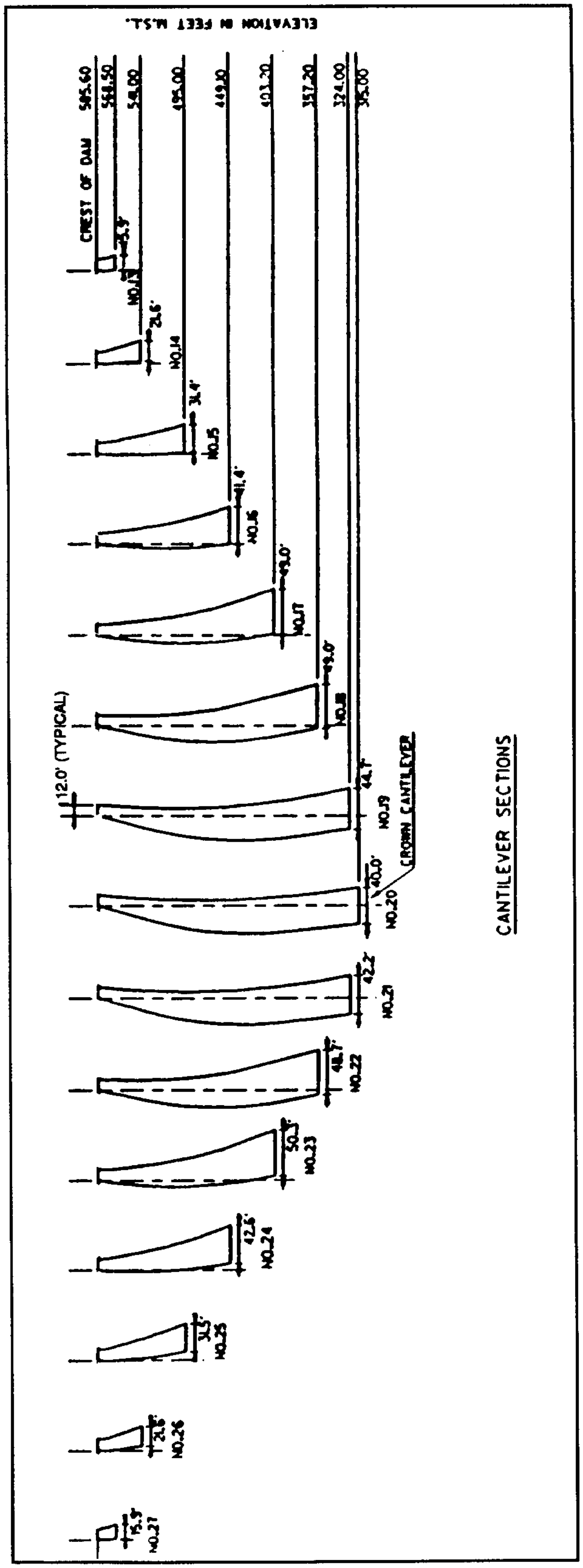


Figure 5-14. Cantilever sections

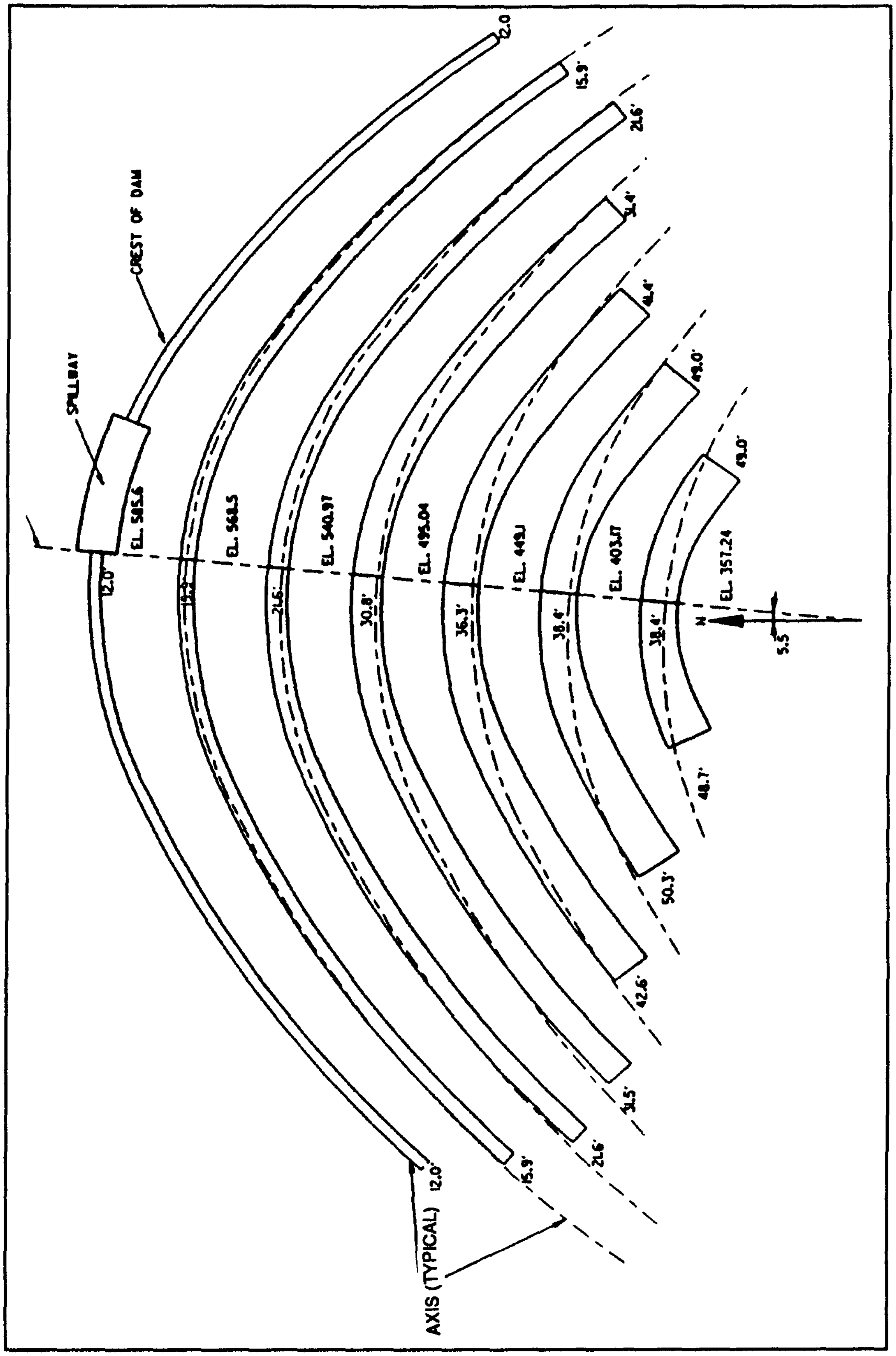


Figure 5-15. Arch sections

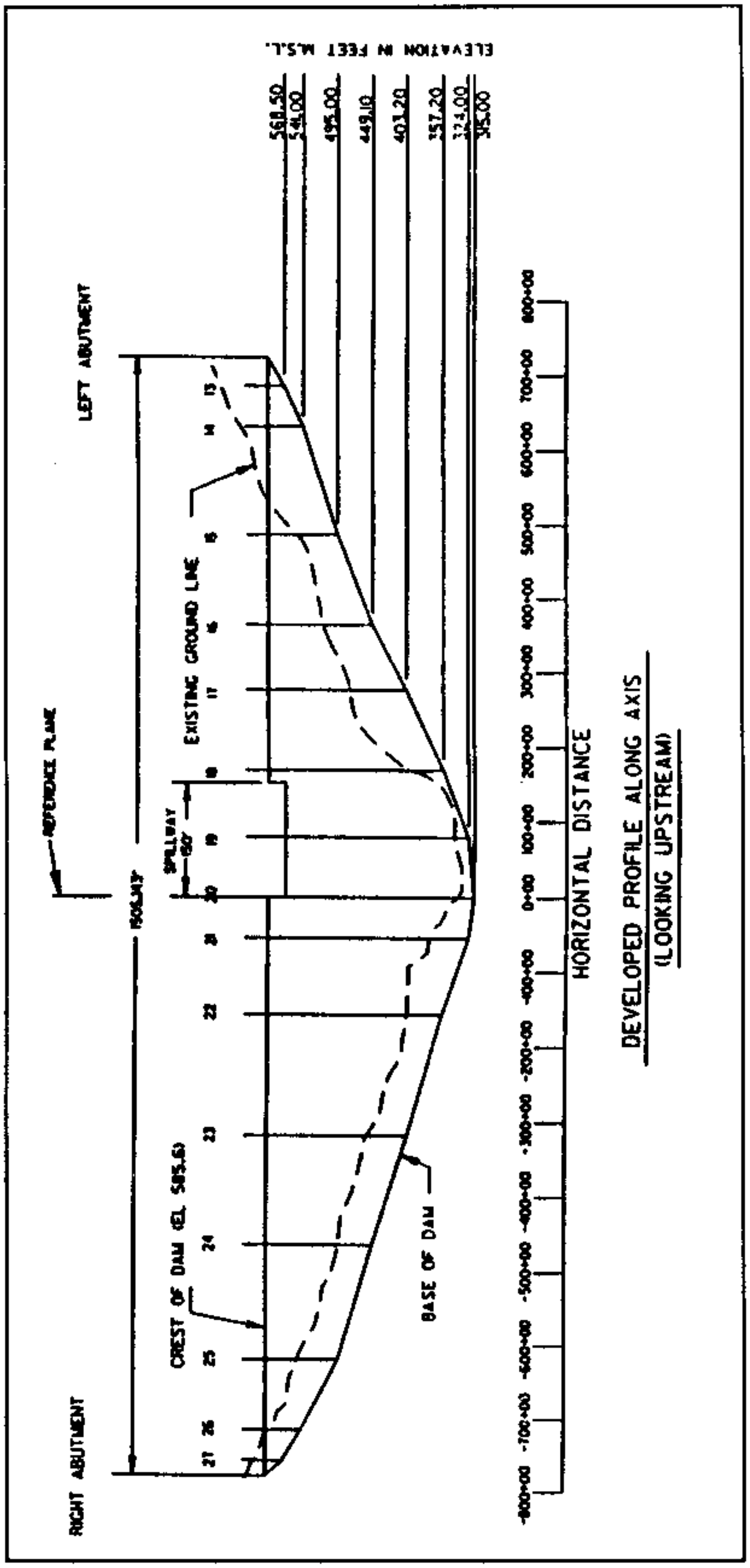


Figure 5-16. Developed profile

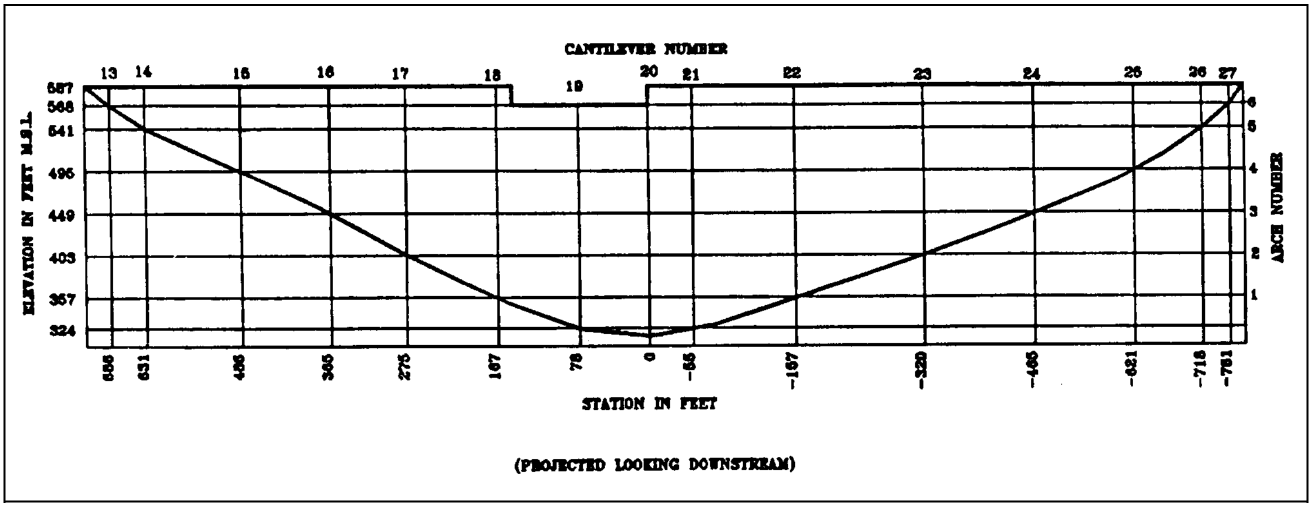


Figure 5-17. ADSAS model

f. ADSAS Model. A plot showing the model of cantilever and arch units generated by ADSAS and the associated cantilever and arch numbering scheme is plotted and shown in the FDM as shown in Figure 5-17.

g. Stress Contours. Contour plots of arch and cantilever stresses on the upstream and downstream faces for all load cases are presented in the FDM as shown in Figure 5-18.

h. Dead Load Stresses. Stresses produced in the ungrouted cantilevers as a result of the construction sequence should be tabulated and presented in the FDM as shown in Figure 5-19.

5-9. Computer-assisted Layouts. The procedures mentioned in this chapter involve manual layout routines using normal drafting equipment. As mentioned, the iterative layout procedure can be quite time consuming. Automated capabilities using desktop computers are currently being developed which enable the structural designer to interactively edit trial layouts while continuously updating plan, plane of centers, and profile views. When an acceptable layout is achieved, the program generates an ADSAS data file which is input into a PC version of ADSAS. In all, developing these tools on a desktop computer allows the structural designer to proceed through the layout process at a faster rate than could be achieved manually, thereby, reducing design time and cost.

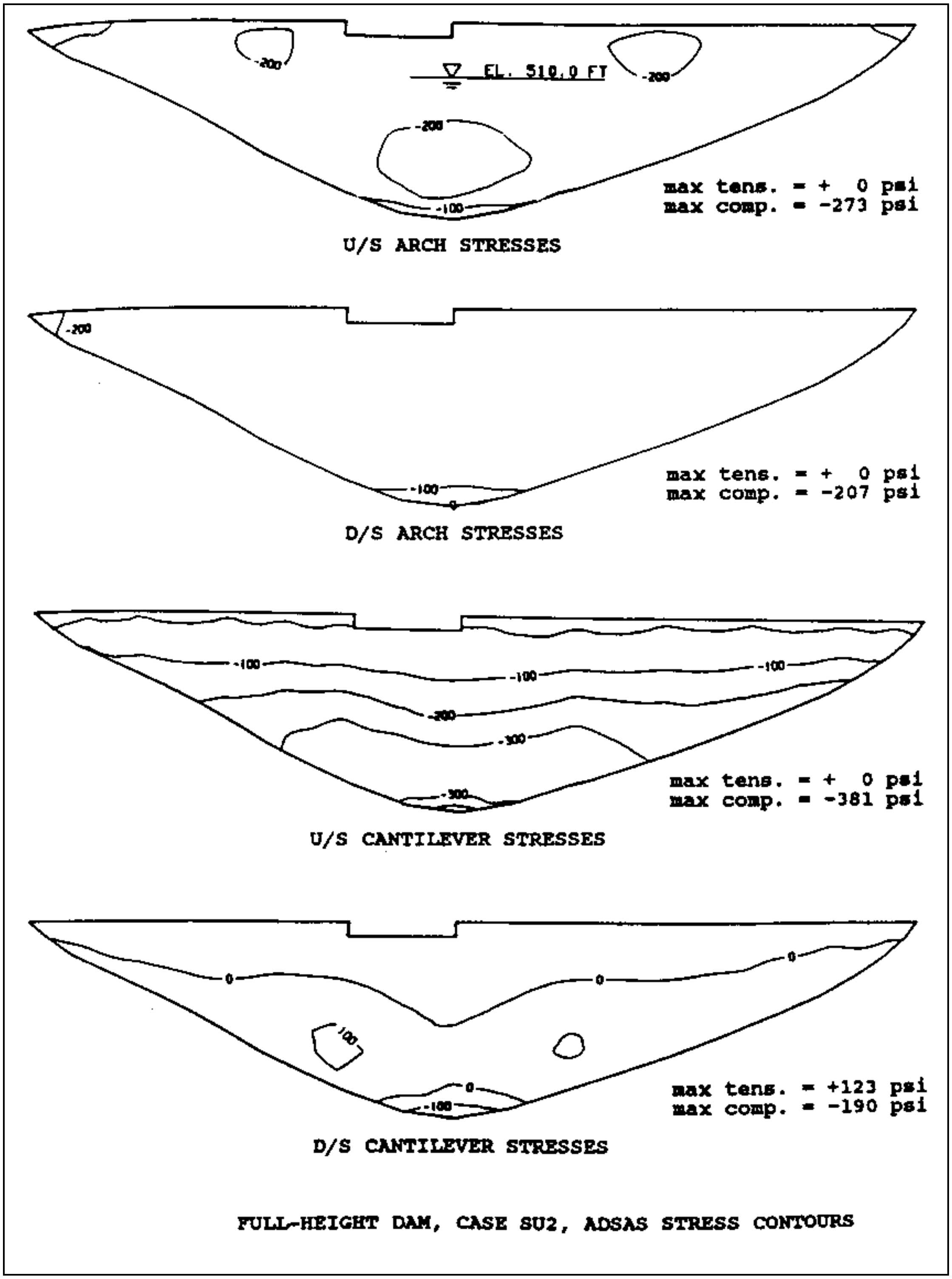


Figure 5-18. ADSAS stress contours

MINIMUM DEAD LOAD STRESSES (IN PSI) BY CANTILEVER

CANTILEVER NUMBER	STRESS BASED ON CONCRETE PLACED ON	ELEVATION OF MINIMUM STRESS	MINIMUM DOWNSTREAM STRESS	UPSTREAM FACE STRESS
20	495.04	314.96	48	-406.
21	540.97	324.00	66.	-459.
22	587.20	357.24	142.	-483.
23	587.20	403.27	111.	-357.
24	587.20	449.10	80.	-265.
25	587.20	495.04	61.	-197.
26	587.20	540.97	18.	-95.
27	587.20	567.20	-2.	-35.
19	540.97	324.00	103.	-482.
18	587.20	357.24	156.	-496.
17	587.20	403.17	89.	-340.
16	587.20	449.10	37.	-227.
15	587.20	495.04	15.	-152.
14	587.20	540.97	5.	-81.
13	587.20	567.20	-6.	-32.

NOTE: MINUS SIGN INDICATES COMPRESSION

Figure 5-19. Dead load stresses

CHAPTER 6

STATIC ANALYSIS

6-1. Introduction. This chapter describes static analysis of concrete arch dams using the FEM. The purpose of FEM analysis is to perform more accurate and realistic analysis by eliminating many assumptions made in the traditional methods. The main advantages of FEM are its versatility and its ability for exploring foundation conditions and representing the more realistic interaction of dam and foundation rock. In particular, nonhomogeneous rock properties, weak zones, clay or gouge seams, and discontinuities in the foundation may be considered in the analysis to evaluate their effects on the stress distribution. The cracked sections or open joints in the structure can be modeled; thrust blocks and the spillway openings in the crest are appropriately included in the mathematical models; and the stresses around the galleries and other openings can be investigated.

6-2. Design Data Required. Design data needed for structural analysis of a concrete arch dam are: Poisson’s ratio, strength and elastic properties of the concrete, Poisson’s ratio and deformation modulus of the foundation rock, unit weight and coefficient of thermal expansion of the concrete, geometric data of the dam layout, geometric data of spillway openings and thrust blocks, operating reservoir and tailwater surfaces, temperature changes within the dam, probable sediment depth in the reservoir, probable ice load, and the uplift pressure. A description of each data type is as follows:

a. Concrete Properties. The material properties of the concrete for use in static analysis are influenced by mix proportions, cement, aggregate, admixtures, and age. These data are not available beforehand and should be estimated based on published data and according to experience in similar design and personal judgment; however, actual measured data should be used in the final analysis as they become available. The concrete data needed for the analysis are:

(1) Sustained modulus of elasticity

(2) Poisson’s ratio

(3) Unit weight

(4) Compressive strength

(5) Tensile strength

(6) Coefficient of thermal expansion

The sustained modulus of elasticity is used in the analyses of the static loads to account for the creep effects. In the absence of long-term test data, a sustained modulus of elasticity equal to 60 to 70 percent of the instantaneous modulus may be used. The standard test method for measurement of the concrete properties is given in Chapter 9.

b. Foundation Properties. The foundation data required for structural analysis are Poisson's ratio and the deformation modulus of the rock supporting the arch dam. Deformation modulus is defined as the ratio of applied stress to resulting elastic plus inelastic strains and thus includes the effects of joints, shears, and faults. Deformation modulus is obtained by in situ tests (Structural Properties, Chapter 9) or is estimated from elastic modulus of the rock using a reduction factor (Von Thun and Tarbox 1971 (Oct)). If more than one material type is present in the foundation, an effective deformation modulus should be used instead. For nonhomogeneous foundations, several effective deformation modulus values may be needed to adequately define the foundation characteristics.

c. Geometric Data. The necessary data for constructing a finite element mesh of an arch dam is obtained from drawings containing information defining the geometry of the dam shape. These include the plan view and section along the reference plane, as shown in Figures 1-5 and 1-6. In practice, arch dams are geometrically described as multicentered arches with their centers varied by elevation in addition to the arch opening angles and radii varying for each side with elevation. Elliptical arch shapes may be approximated for the various elevations by three-centered arches including central segments with shorter radii and two outer segments with equal but longer radii. The basic geometric data of a multicentered dam at each elevation for the upstream and downstream faces are as follows:

(1) Radius of central arcs

(2) Radius of outer arcs

(3) Angles to point of compounding curvatures

(4) Angles to abutments

(5) Location of centers of central arcs

Preparation of finite element mesh data from these geometric data is very time consuming because most general-purpose finite element programs cannot directly handle these data or the similar ADSAS input data; however, GDAP (Ghanaat 1993a), a specialized arch dam analysis program, can automatically generate coordinates of all nodal points, element data, element distributed loads, and the nodal boundary conditions from such limited geometric data or even directly from ADSAS input data for any arch dam-foundation system. Geometric data for modeling thrust blocks, spillway openings, and other structural features are obtained directly from the associated design drawings.

d. Static Loads. The basic loads contributing to the design or safety analysis of arch dams are gravity, reservoir water, temperature changes, silt, ice, uplift, and earthquake loads. The data needed to specify each individual static load are described in this section. Earthquake loads and their effects are discussed in Chapter 7, and the various load combinations are presented in Chapter 4.

(1) Gravity Loads. Gravity loads due to weight of the material are computed from the unit weight and geometry of the finite elements. The dead weight may be applied either to free-standing cantilevers without arch action

to simulate the construction process or to the monolithic arch structure with all the contraction joints grouted. Although the first assumption usually is more appropriate, a combination of the two is more realistic in situations where the vertical curvature of the cantilevers is so pronounced that it is necessary to limit the height of free-standing cantilevers by grouting the lower part of the dam. In those cases, a gravity load analysis which closely follows the construction sequence is more representative. The weight of the appurtenant structures that are not modeled as part of the finite element model but are supported by the dam, if significant, are input as external concentrated loads and are applied to the supporting nodal points.

(2) Reservoir Water. Most finite element programs such as the GDAP and SAP-IV handle hydrostatic loads as distributed surface loads. The surface loads are then applied to the structure as concentrated nodal loads. Therefore, hydrostatically varying surface pressure can be specified by using a reference fluid surface and a fluid weight density as input.

(3) Temperature. Temperature data needed in structural analysis result from the differences between the closure temperature and concrete temperature expected in the dam during its operation. Temperature changes include high and low temperature conditions and usually vary: by elevation, across the arch, and in the upstream-downstream direction. Temperature distribution in the concrete is determined by temperature studies (Chapter 8) considering the effects of transient air and water temperatures, fluctuation of reservoir level, and the solar radiation. The nonlinear temperature distribution calculated in these studies is approximated by straight line distribution through the dam thickness for the use in structural analysis performed in using shell elements. However, if several solid elements are used through the thickness, a nonlinear temperature distribution can be approximated.

(4) Silt. Arch dams often are subjected to silt pressures due to sedimentary materials deposited in the reservoir. The saturated silt loads are treated as hydrostatically varying pressures acting on the upstream face of the dam and on the valley floor. A silt reference level and the weight density of the equivalent fluid are needed to specify the silt pressures.

(5) Ice. Ice pressure can exert a significant load on dams located at high altitudes and should be considered as a design load when the ice cover is relatively thick. The actual ice pressure is very difficult to estimate because it depends on a number of parameters that are not easily available. In that case, an estimate of ice pressure as given in Chapter 4 may be used.

(6) Uplift. The effects of uplift pressures on stress distribution in thin arch dams are negligible and, thus, may be ignored; however, uplift can have a significant influence on the stability of a thick gravity-arch dam and should be considered in the analysis. For more discussion on the subject, refer to "Theoretical Manual for Analysis of Arch Dams" (Ghanaat 1993b).

6-3. Method of Analysis. The static analysis of an arch dam should be based on the 3-D FEM. The FEM is capable of representing the actual 3-D behavior of an arch dam-foundation system and can handle any arbitrary geometry of the dam and valley shape. Furthermore, the method can account for a variety of loads and is equally applicable to gravity arch sections as well as to slender and doubly curved arch dam structures.

a. The FEM is essentially a procedure by which a continuum such as an arch dam structure is approximated by an assemblage of discrete elements interconnected only at a finite number of nodal points having a finite number of unknowns. Although various formulations of the FEM exist today, only the displacement-based formulation which is the basis for almost all major practical structural analysis programs is described briefly here. The displacement-based FEM is an extension of the displacement method that was used extensively for the analysis of the framed and truss type structures before the FEM was developed (Przemieniecki 1968). Detailed formulations of the FEM are given by Zienkiewicz (1971), Cook (1981), and Bathe and Wilson (1976). Application of the method to the analysis of arch dams is presented in the "Theoretical Manual for Analysis of Arch Dams" (Ghanaat 1993b). Following is an outline of the finite element computer analysis for static loads, as a sequence of analytical steps:

(1) Divide the dam structure and the foundation rock into an appropriate number of discrete subregions (finite elements) connected at joints called nodal points. For a discussion of mesh density, see paragraph 6-4.

(2) Compute the stiffness matrix of each individual element according to the nodal degrees of freedom and the force-displacement relationships defining the element.

(3) Add the stiffness matrices of the individual elements to form the stiffness matrix of the complete structure (direct stiffness method).

(4) Define appropriate boundary conditions and establish equilibrium conditions at the nodal points. The resulting system of equations for the assembled structure may be expressed as:

$$ku = p \tag{6-1}$$

where

k = stiffness matrix
u = displacement vector
p = load vector

(5) Solve the system of equations for the unknown nodal displacements u.

(6) Calculate element stresses from the relationship between the element strains and the nodal displacements assuming an elastic strain-stress relationship.

b. Most general-purpose finite element computer programs follow these above analytical steps for static structural analysis, but their applicability to arch dams may be judged by whether they have the following characteristics:

(1) An efficient graphics-based preprocessor with automatic mesh generation capabilities to facilitate development of mathematical models and to check the accuracy of input data.

(2) Efficient and appropriate finite element types for realistic representation of the various components of the dam structure.

(3) Efficient programming methods and numerical techniques appropriate for the solution of large systems with many degrees of freedom.

(4) Postprocessing capabilities providing graphics for evaluation and presentation of the results.

c. SAP-IV (Bathe, Wilson, and Peterson 1974) and GDAP (Ghanaat 1993a) are two widely used programs for the analysis of arch dams. These programs are briefly described here. SAP-IV is a general-purpose finite element computer program for the static and dynamic analysis of linearly elastic structures and continua. This program has been designed for the analysis of large structural systems. Its element library for dam analysis includes eight-node and variable-number-node, 3-D solid elements. The program can handle various static loads including hydrostatic pressures, temperature, gravity due to weight of the material, and concentrated loads applied at the nodal points. However, the program lacks pre- and postprocessing capabilities. Thus, finite element meshes of the dam and foundation must be constructed manually from the input nodal coordinates and element connectivities. Also, the computed stress results are given in the direction of local or global axes and cannot be interpreted reliably unless they are transformed into dam surface arch and cantilever stresses by the user.

d. GDAP has been specifically designed for the analysis of arch dams. It uses the basic program organization and numerical techniques of SAP-IV but has pre- and postprocessing capabilities in addition to the special shell elements. The thick-shell element of GDAP, which is represented by its mid-surface nodes, uses a special integration scheme that improves bending behavior of the element by reducing erroneous shear energy. The 16-node shell is the other GDAP special dam analysis element; this retains all 16-surface nodes and uses incompatible modes to improve the bending behavior of the element. In addition to the shell elements, eight-node solid elements are also provided for modeling the foundation rock. The preprocessor of GDAP automatically generates finite element meshes for any arbitrary geometry of the dam and the valley shape, and it produces various 3-D and 2-D graphics for examining the accuracy of mathematical models. The postprocessor of GDAP displays nodal displacements and provides contours of the dam face arch and cantilever stresses as well as vector plots of the principal stresses acting in the faces.

e. Other general-purpose FEM programs, such as ABAQUS (Hibbitt, Karlsson, and Sorenson 1988), GTSTRUDL (Georgia Institute of Technology 1983), etc., can also be used in the analysis of arch dams. Special care should be used to assure that they have the characteristics identified in paragraph 6-3b. Also, the stress results from general-purpose FEM programs may be computed in local or global coordinates and, therefore, may need to be translated into surface arch and cantilever stresses by the user prior to postprocessing.

6-4. Structural Modeling. Arch dams are 3-D systems consisting of a concrete arch supported by flexible foundation rock and impounding a reservoir of water. One of the most important requirements in arch dam analysis is to develop accurate models representative of the actual 3-D behavior of the system. A typical finite element idealization of a concrete arch dam and its foundation rock is shown in Figure 6-1. This section presents general guidelines on structural modeling for linear-elastic static analysis of single arch dams. The guidelines aim to provide a reasonable compromise between the accuracy of the analysis and the computational costs. They are primarily based on the results of numerous case studies and not on any rigorous mathematical derivation. The procedures and guidelines for developing mathematical models of various components of an arch dam are as follows:

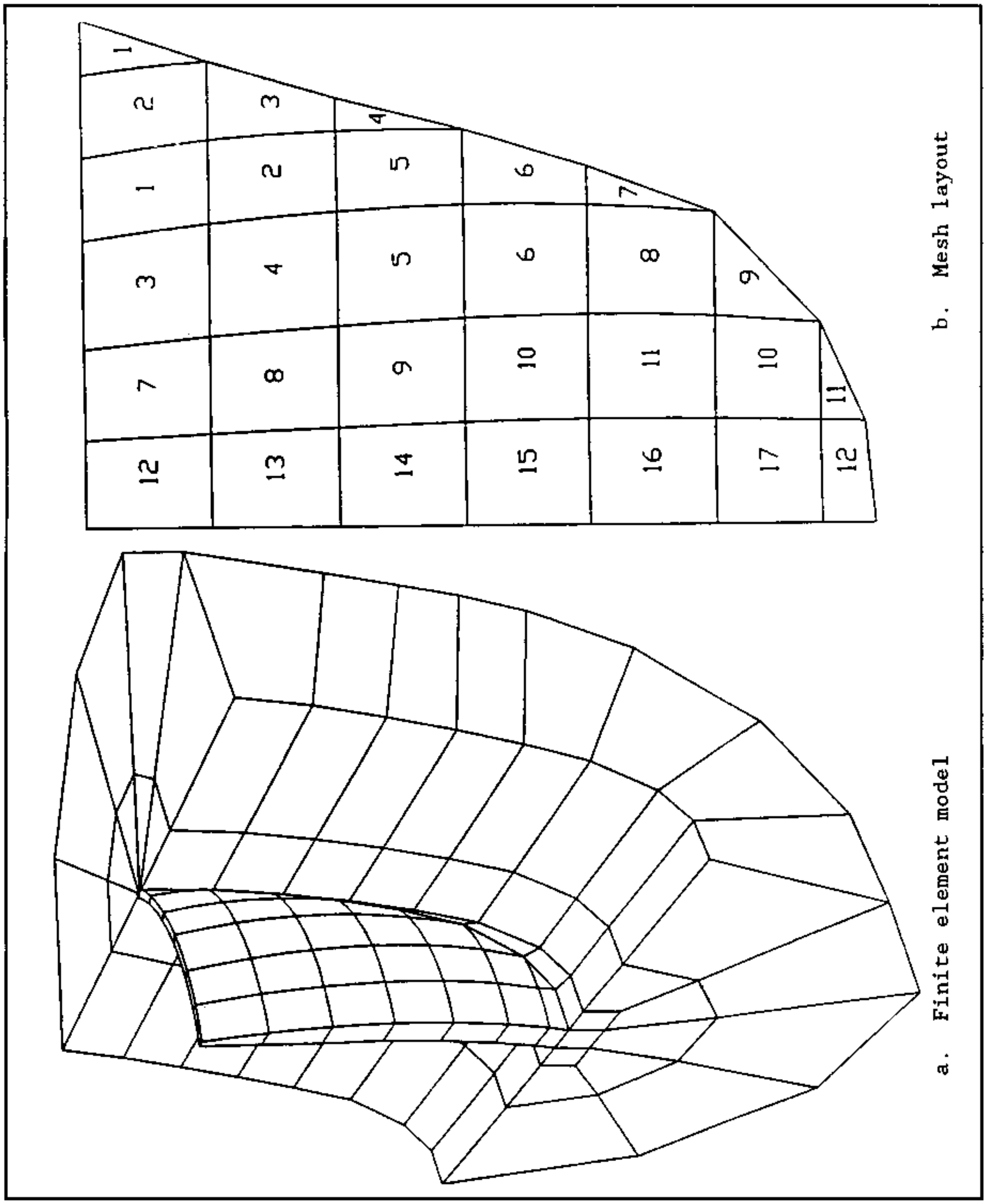
a. Dam Model. An appropriate finite element mesh for an arch dam can only be achieved by careful consideration of the dam geometry and the type of analysis for which the dam is modeled. For example, the finite element model of a double-curvature thin-shell structure differs from the model of a thick gravity-arch section. Furthermore, a structural model developed solely for a linear-elastic analysis generally is not appropriate for a nonlinear analysis.

(1) Number of Element Layers. Arch dam types may be divided, according to the geometry of their cross sections, into thin, moderately thin, and thick gravity-arch sections. Table 6-1 identifies each of these types with regard to crest thickness (t_c) and base thickness (t_b), each expressed as a ratio to the height (H). Also shown is the ratio of base-to-crest thickness. Each of these dam types may be subject to further classification based on the geometry of arch sections as described in Chapter 1. The GDAP element library contains several elements for modeling the dam and foundation, as described previously and shown in Figures 6-2, 6-3, and 6-4. The body of a thin arch dam, usually curved both in plan and elevation, is best represented by a combination of special-purpose shell elements available in the GDAP program (Figures 6-2b, 6-4c and d). The general 3-D solid element shown in Figure 6-4b, which may have from 8 to 21 nodes, can also be used, but these are not as accurate as the GDAP shell elements in representing bending moments and shear deformations of thin shell structures. In either case, a single layer of solid elements which use quadratic displacement and geometry interpolation functions in the dam face directions and linear interpolation in the dam thickness direction is sufficient to accurately represent the body of the dam (Figure 6-1). These finite elements are discussed in more detail in the "Theoretical Manual for Analysis of Arch Dams" (Ghanaat 1993b).

(a) Moderately thin arch dams are modeled essentially similar to the thin arch dams, except that 3-D solid elements should be used near the base and the abutment regions where the shell behavior assumption becomes invalid due to excessive thickness of the arch.

(b) Gravity-arch dams should be modeled by two or more layers of solid elements in the thickness direction depending on their section thickness. Any of the solid elements shown in Figures 6-4a, b, or d may be used to model the dam. It is important to note that multilayer element meshes are essential to determine a detailed stress distribution across the thickness and to provide additional element nodes for specifying nonlinear temperature distributions.

(2) Size of the Dam Mesh. There are no established rules for selecting an optimum mesh size for subdividing an arch dam in the surface directions. The best approach, however, is to define and analyze several meshes of different element types and sizes and then select the one that is computationally



a. Finite element model

b. Mesh layout

Figure 6-1. Finite element model of Morrow Point Dam and foundation

TABLE 6-1

Arch Dam Types

	t_c/H	t_b/H	t_b/t_c
Thin arch	0.025-0.05	0.09-0.25	2.9-5
Moderately thin	0.025-0.05	0.25-0.4	5-10
Thick gravity-arch	0.05 -0.10	0.5 -1.0	8-15

efficient and provides reasonably accurate results. The main factors to consider in choosing the mesh include the size and geometry of the dam, type of elements to be used, type and location of spillway, foundation profile, as well as dynamic characteristics of the dam, and the number of vibration modes required in the subsequent earthquake analysis. The size of the finite elements should be selected so that the mesh accurately matches the overall geometry, the thickness, and the curvature of the dam structures. As the dam curvature increases, smaller elements are needed to represent the geometry. The element types used to model a dam affect not only the required mesh size but greatly influence the results. For example, idealization of arch dams with flat face elements requires the use of smaller elements and, thus, a larger number of them, and yet such elements can not reproduce the transverse shear deformations through the dam which may not be negligible. On the other hand, the same dam can be modeled with fewer curved thick-shell elements such as those available in GDAP and thus obtain superior bending behavior and also include the transverse shear deformations. Figure 6-2 shows an example of three finite element meshes of Morrow Point Arch Dam with rigid foundation rock. Downstream deflections of the crown cantilever due to hydrostatic loads (Figure 6-5) indicate that normal and fine meshes of shell elements provide essentially identical results, and the coarse mesh of shell elements underestimates the deflections by less than 1 percent at the crest and by less than 10 percent at lower elevations. Similar results were obtained for the stresses but are not shown here. This example indicates that the normal mesh size provides accurate results and can be used in most typical analysis. If desired, however, the coarse mesh may be used in preliminary analyses for reasons of economy. For the thick-shell elements used in this example, various parameters such as the element length along the surface (a), the ratio of the thickness to the length (t/a), and the ratio of the length to the radius of curvature ($\phi = a/R$) for the coarse and normal meshes are given in Table 6-2 as a reference. These data indicate that the GDAP shell elements with an angle of curvature less than 20 degrees and a length equal or less than 150 feet, provide sufficient accuracy for practical analysis of arch dams with simple geometry and size comparable to that of Morrow Point Dam. For other arch dams with irregular foundation profile, or with attached spillway, or when the lower-order solid elements are used, a finer mesh than that shown in Table 6-2 may be required.

b. Foundation Model. An ideal foundation model is one which extends to infinity or includes all actual geological features of the rock and extends to

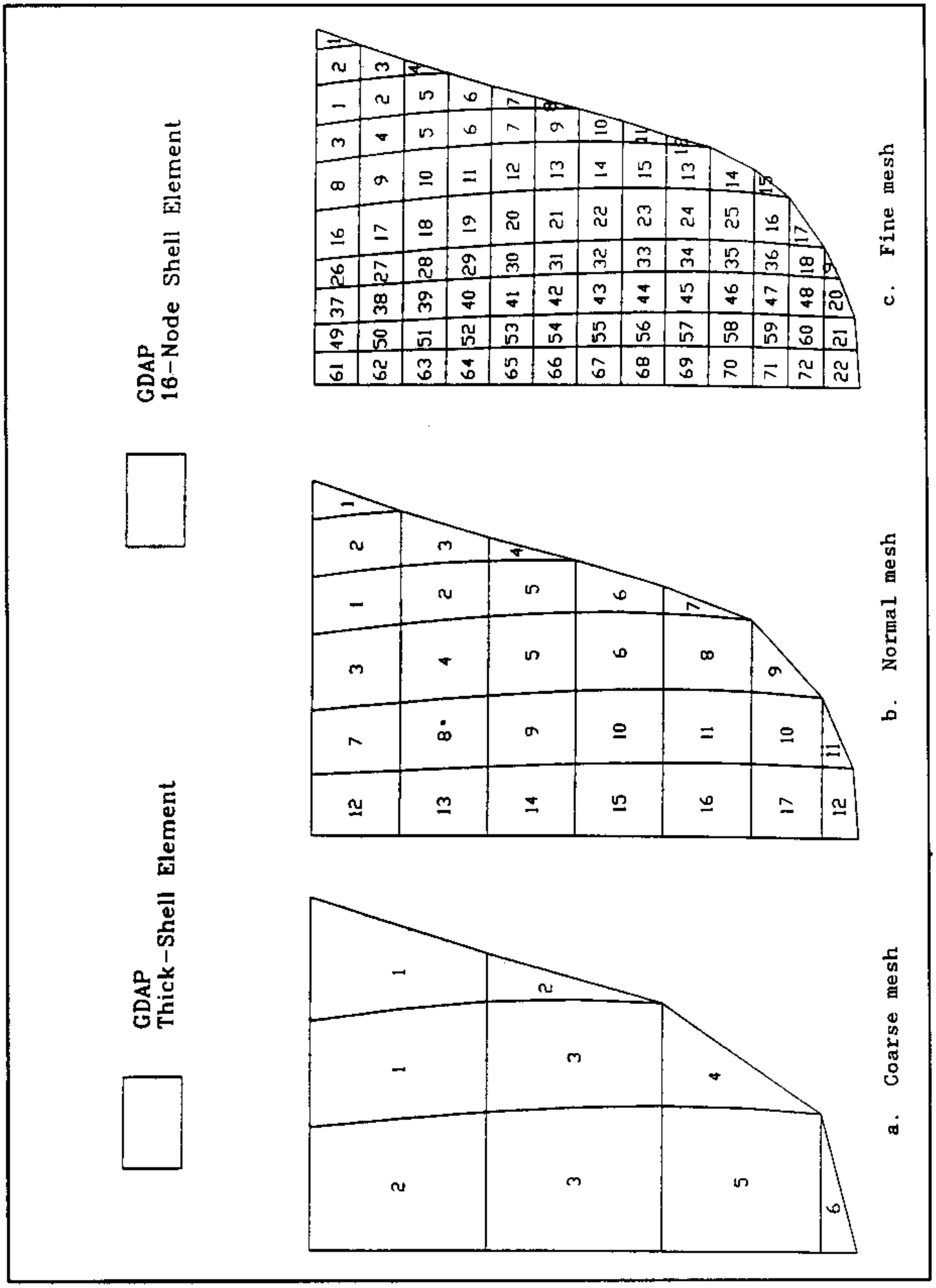


Figure 6-2. Alternative meshes for Morrow Point Dam

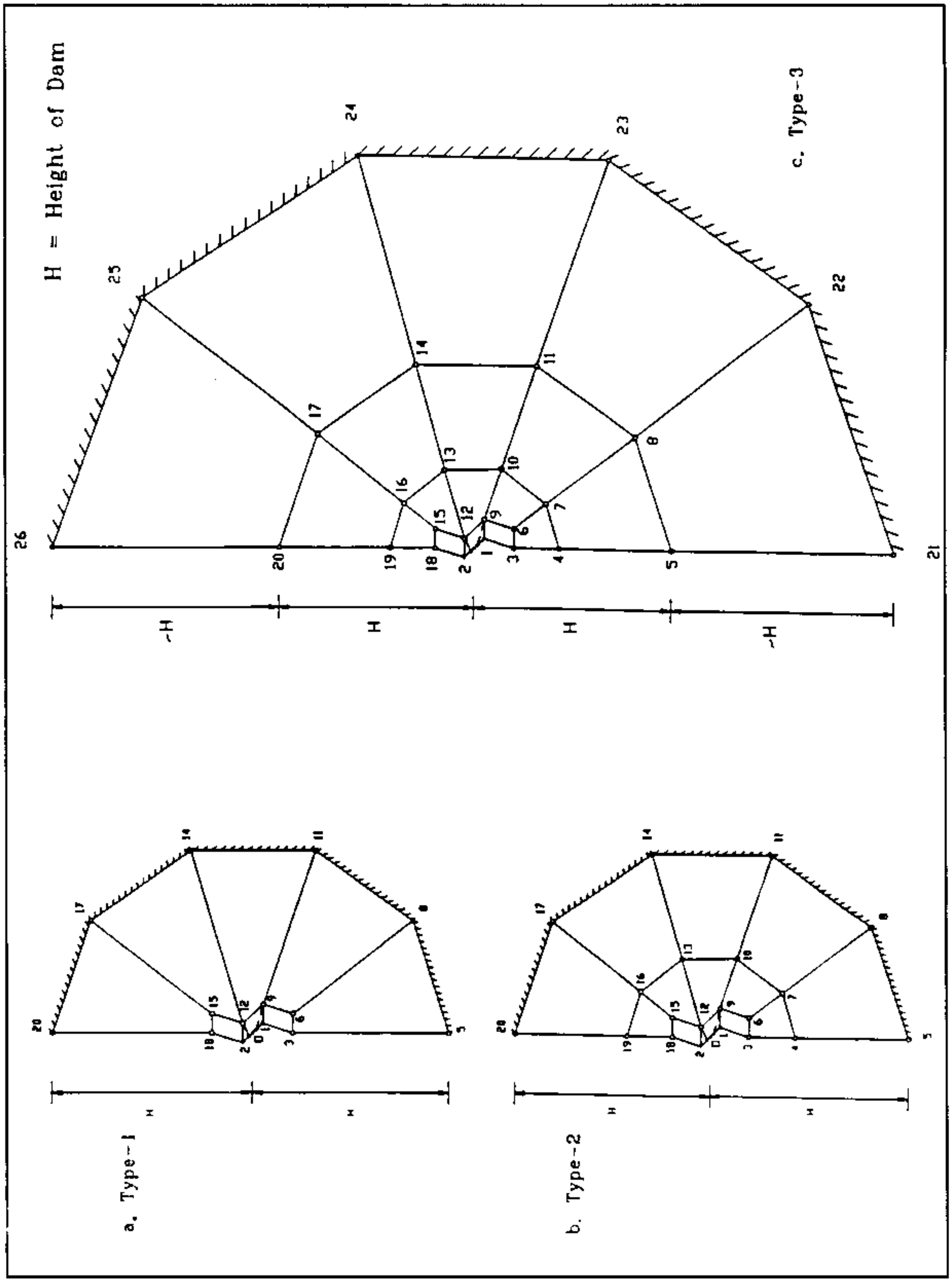


Figure 6-3. GDAP foundation mesh types

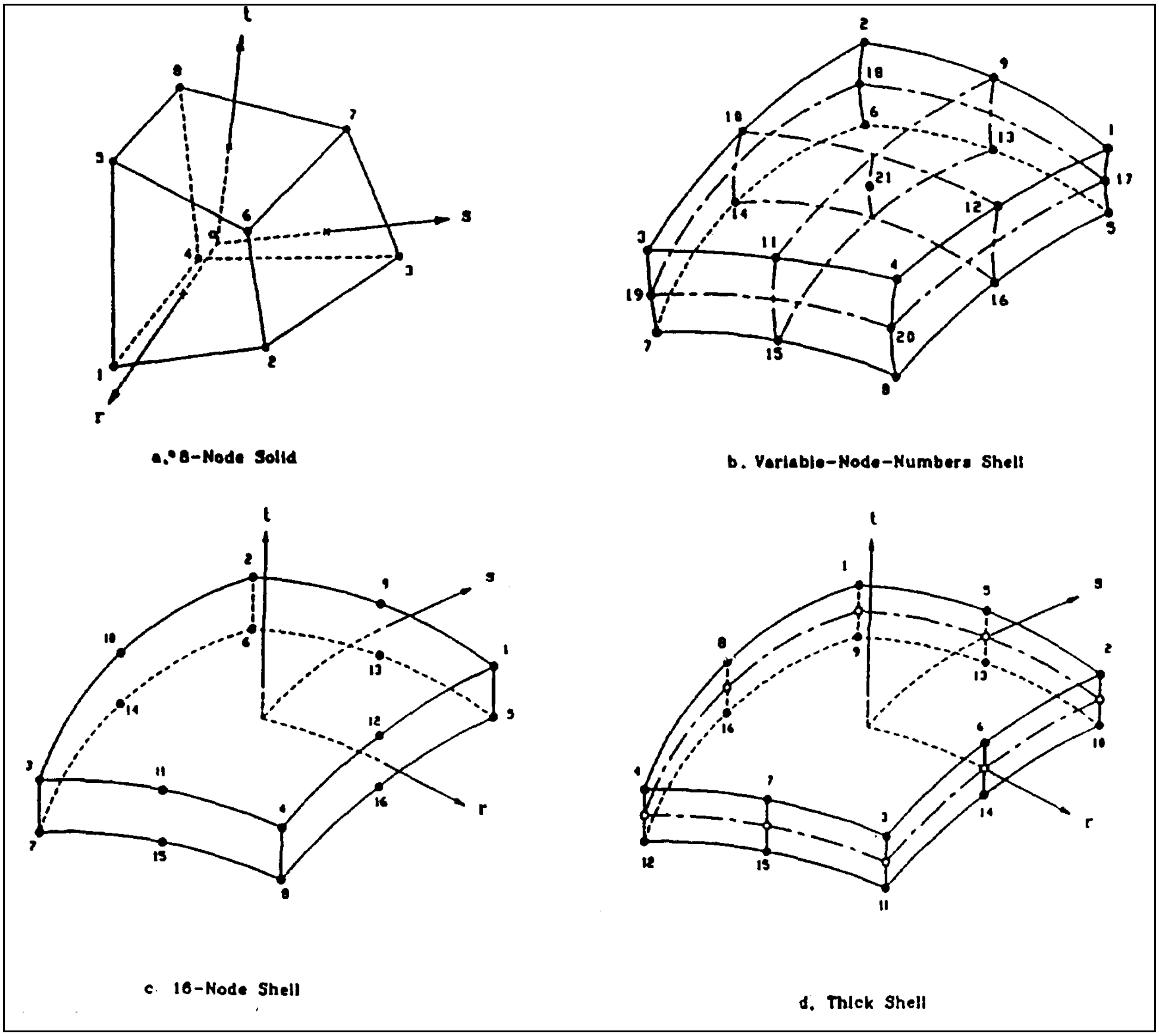


Figure 6-4. Shell and 3-D elements for arch dams

a very large distance where boundary effects on the stresses in the dam become negligible. In practice, however, these idealized models are not possible because analytical techniques to deal with infinite foundation models are not yet sufficiently developed, and very extensive models are computationally prohibitive, even if the necessary geological data were available. Instead, a simplified foundation model is used which extends a sufficiently large distance that boundary effects are insignificant; the effects of the geological formation are partly accounted for by using modulus of deformation rather than the modulus of elasticity of the rock. In general, the geometry of the rock supporting an arch dam is completely different for different dams and cannot be represented by a single rule; however, simplified prismatic foundation models available in the GDAP program (Figure 6-1) provide adequate models that can conveniently be adapted to different conditions. The foundation mesh types available in the GDAP are shown in Figure 6-3. All three meshes are

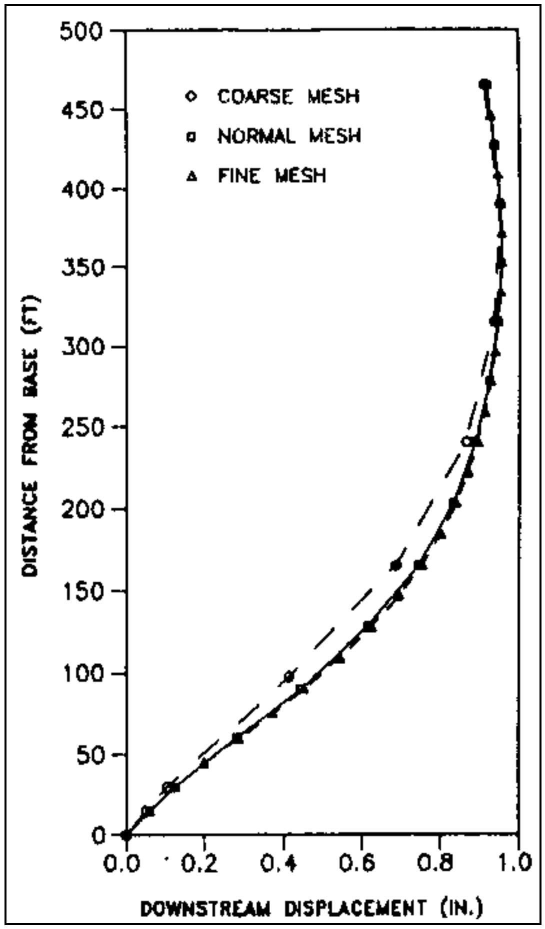


Figure 6-5. Crown section displacements of Morrow Point Dam for alternative meshes

TABLE 6-2

Element Mesh Parameters

Mesh	a ft	t/a crest	t/a base	ϕ
Coarse	150	0.08	0.34	20
Normal	75	0.15	0.70	10

constructed on semicircular planes cut into the canyon walls and oriented normal to the rock-concrete interface as indicated in Figure 6-1a; they differ only with respect to the extent of the rock and the number of elements in each semicircular plane. Eight-node solid elements with anisotropic material properties (Figure 6-4a) are most commonly used for modeling the foundation rock.

The foundation mesh is arranged so that smaller elements are located adjacent to the dam-foundation contact surface and the elements become larger toward the boundaries of the model. The size of elements used near the interface is controlled by the dam thickness, and the size of the larger elements depends on the extent of foundation mesh and the number of elements to be used in each section.

(1) Effects of Foundation Deformability. The importance of foundation interaction on the displacements and stresses resulting from loading an arch dam has long been recognized. The results of a parametric study of Morrow Point Dam, presented in Figures 6-6 through 6-8, demonstrate qualitatively the relative importance of the foundation modulus on the dam response. Three values of the rock modulus were considered: (a) rigid, (b) the same modulus as the concrete, and (c) one-fifth (1/5) the modulus of concrete. The analyses were made only for hydrostatic loads, and the effect of water load acting on the flexible foundation at the valley floor and on the flanks was also investigated. Figure 6-6 shows the deformation patterns while Figure 6-8 compares the arch and cantilever stresses along the crown cantilever section. Deformations clearly are strongly affected by the rock modulus. The rotation of foundation rock, caused by the reservoir water, results in a slight rotation of the dam section in the upstream direction which is more pronounced for weaker rocks. Stresses also are considerably affected by foundation flexibility as compared with the rigid foundation assumption and are further increased by the weight of the impounded water which causes deformations of the foundation rock at the valley floor and flanks. It is important to realize that actual foundations are seldom uniform and may have extensive weak zones. In such cases different values of rock modulus should be assigned to different zones so that the variability effects may be assessed.

(2) Size of the Foundation-Rock Region. To account for the flexibility effects of the foundation rock, an appropriate volume of the foundation should be included in the dam-foundation model to be analyzed; however, the amount of flexibility that is contributed by the foundation rock in actual field conditions has not been established. Larger foundation meshes can provide greater flexibility; however, if more finite elements are used to subdivide the foundation rock, greater data preparation and computational efforts are required. Moreover, the increased flexibility also can be obtained by using a reduced foundation modulus. Therefore, the foundation idealization models presented in Figure 6-3 may be sufficient to select the minimum mesh extent (i.e., radius of semicircle R_f) which adequately represents the foundation flexibility effects. In static analysis, flexibility of foundation affects displacements and stresses induced in the dam. For practical analysis, the minimum R_f is selected as a distance beyond which increasing R_f has negligible effects on the displacements and stresses in the dam. The static displacements along the crown cantilever of Morrow Point Dam for two concrete to rock modulus ratios and for three foundation mesh types are shown in Figure 6-7. These results and the stress results (not shown here) suggest that foundation mesh type-1 is adequate for most practical analyses and especially for foundations in which the rock modulus is equal to or greater than the concrete modulus. For very flexible foundation rocks, however, mesh type-3 with an R_f equal to two dam heights should be used.

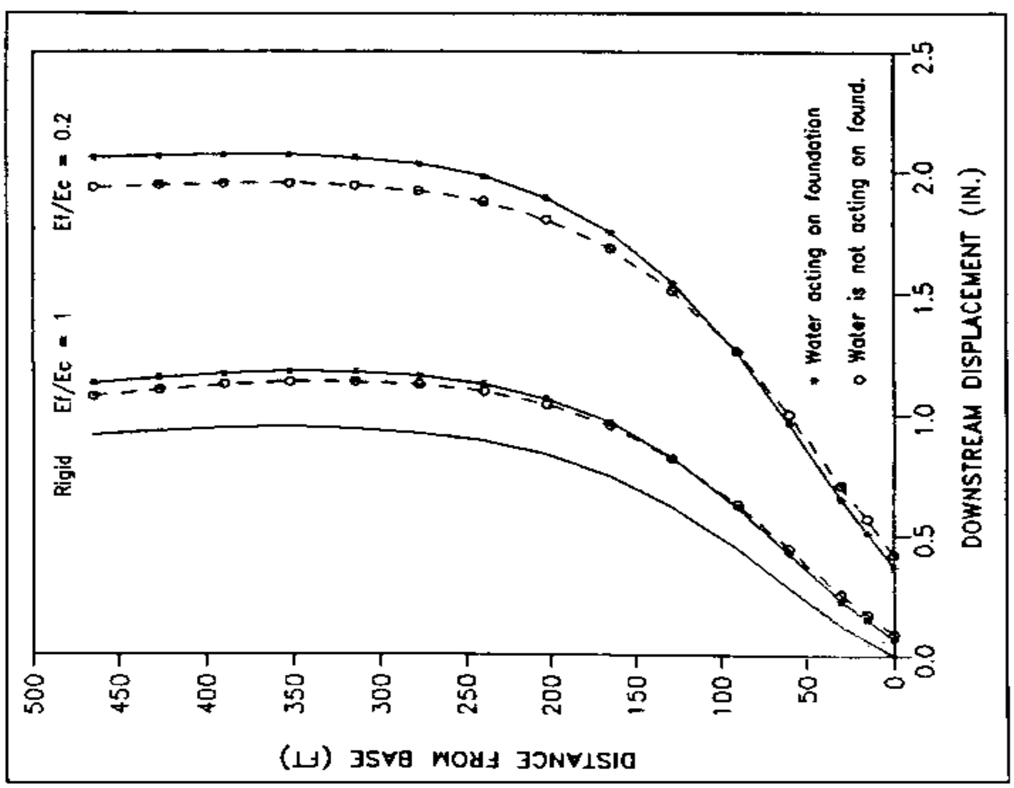


Figure 6-6. Crown section displacements of Morrow Point Dam for different foundation-to-concrete modulus ratios with and without water acting on valley floor

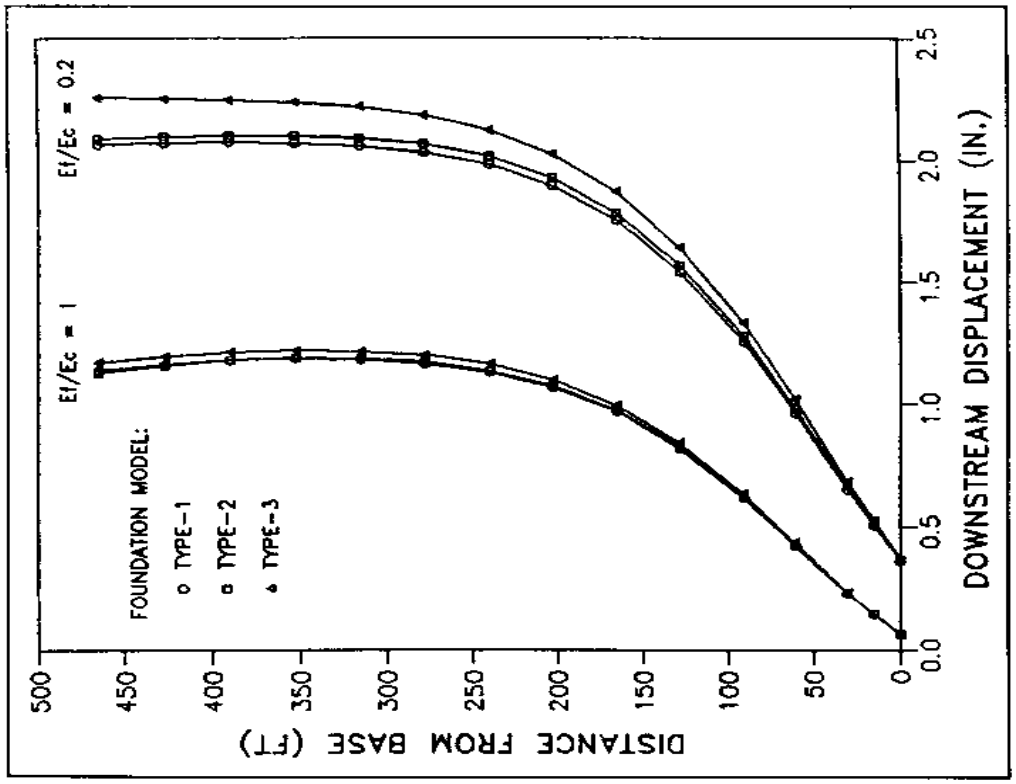


Figure 6-7. Crown section displacements of Morrow Point Dam for two foundation-to-concrete modulus ratios and for three foundation mesh types

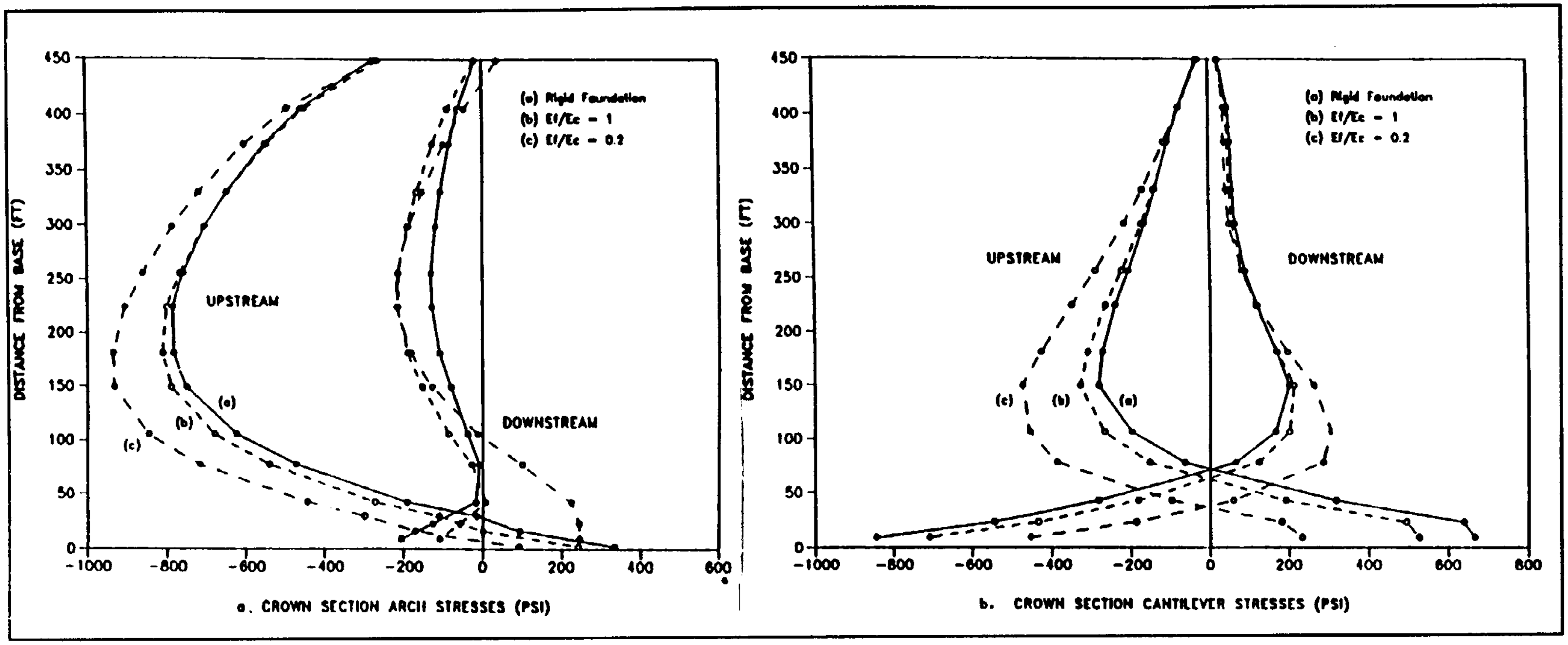


Figure 6-8. Crown section arch and cantilever stresses of Morrow Point Dam for different foundation-to-concrete modulus ratios with (c & d) and without (a & b) water loads acting on valley floor (Continued)

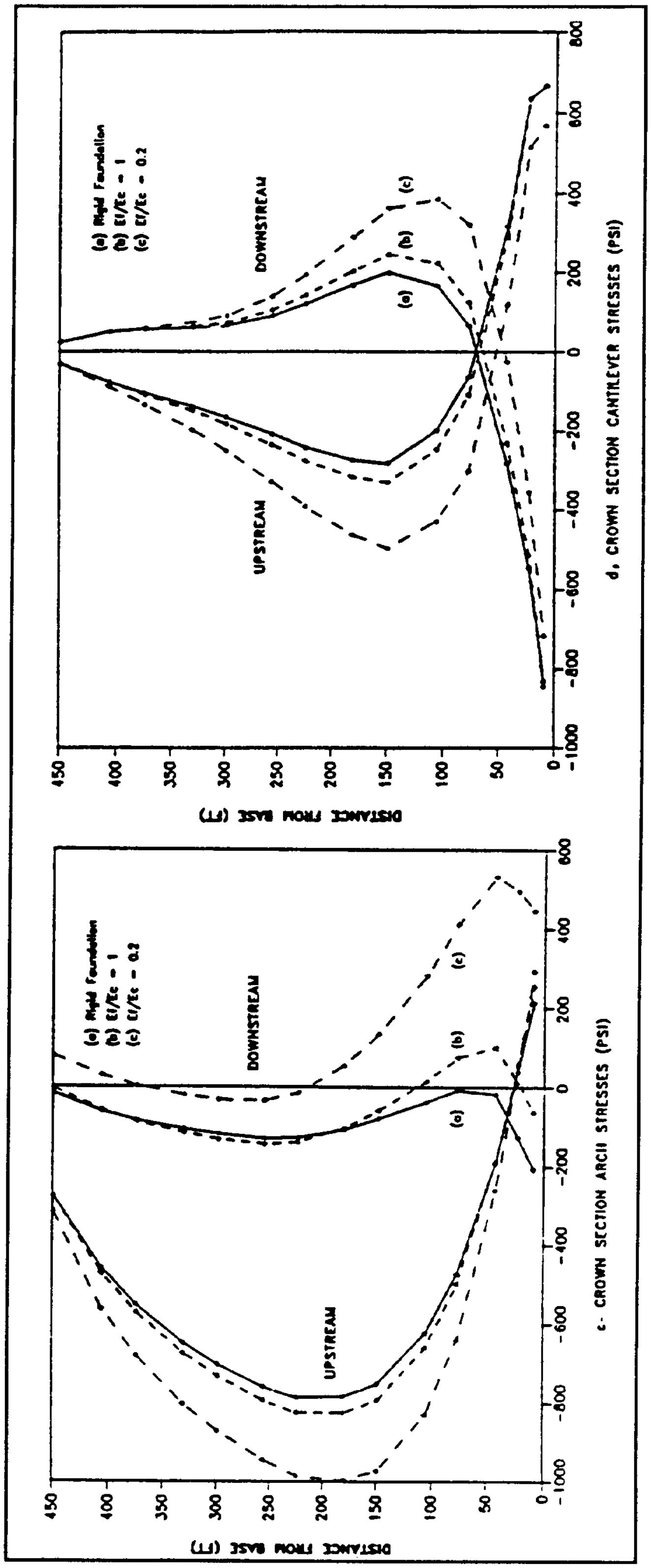


Figure 6-8. (Concluded)

c. Appurtenant Structures. All modern dams include a number of appurtenant structures and devices such as thrust blocks, spillways, galleries, and other openings. The effects of such appurtenances, if significant, should be considered in the analysis by including them in the finite element model of the dam structure.

(1) Thrust Blocks. Thrust blocks are often used as an artificial abutment where the foundation rock does not extend high enough to support the arches. Their main function is to resist the forces transmitted by the upper arches and transfer them to sound foundation rock at their base. They are a critical component of an arch dam design and should be included appropriately as part of the finite element model of the dam-foundation system. Thrust blocks may be adequately modeled using 8-node elements or any variation of 8-to-20 node solid elements shown in Figures 6-4a and 6-4b. Several element layers are usually required to match the arch mesh at the junction and to account for excessive thickness of the thrust block.

(2) Spillways, Galleries, and Other Openings. Arch dams may be designed to accommodate spillways and various other openings such as galleries, sluiceways, and river outlets. Stresses usually tend to concentrate excessively in the area of such openings, and care should be taken to reduce their effects. The large cuts made at the crests of arch dams to provide openings for overflow spillways should be included in the finite element model of the dam structure in order to assess their effects on the stress distribution. If necessary, the design of the dam should be modified to transfer load around the opening in the crest or to proportion the dam thickness to reduce the resulting stress concentrations. Spillways provided by tunnels or side channels that are independent of the arch dam are analyzed and designed separately; thus, they are not included in the finite element model of the dam.

(3) Other Openings Such as the Galleries, Sluiceways, and River Outlets. These openings introduce a local disturbance in the prevalent stress field and, in general, weaken the structure locally; however, the size of these openings usually does not have a significant influence on the overall stiffness of the dam structure, and their effect on the stress distribution may be ignored if adequate reinforcing is provided to carry the forces around the openings. Therefore, such openings need not be considered in the finite element mesh provided that the openings are small and adequately reinforced.

6-5. Presentation of Results. An important aspect of any finite element analysis is that of selecting and presenting essential information from the extensive results produced. It is extremely helpful to have the results presented in graphical form, both for checking and evaluation purposes. The results should contain information for the complete structure to make a judgement regarding the dam safety, as well as to determine whether the boundary locations are suitable or whether there are inconsistencies in the stress distribution.

a. The basic results of a typical static analysis of an arch dam consist of nodal displacements and element stresses computed at various element locations. As a minimum, nodal displacements and surface stresses for the design load combinations specified in Chapter 4 should be presented. Additional displacement and stress results due to the individual load pattern are also desirable because they provide basic information for interpretation of the indicated dam behavior.

b. Nodal displacements are computed in most computer analyses and are directly available. They are simply presented as deflected shapes across selected arches and cantilevers or for the entire dam structure in the form of 3-D plots. However, consideration should be given to whether the displacements should be indicated in global (x,y) coordinates, or in terms of radial and tangential components for each surface node. The stresses usually are computed with respect to a global coordinate system but they should be transformed to surface arch, cantilever, and principal stress directions to simplify their interpretation. The arch and cantilever stress quantities usually are plotted as stress contours on each dam face, while the principal stresses on each face are presented in the form of vector plots as shown in Figure 6-9. In addition, plots of the arch and cantilever stresses determined across the upper arch section and along the cantilever sections are desirable for further detailed study of the stresses.

6-6. Evaluation of Stress Results.

a. Evaluation of the stress results should start with careful examination of the dam response to assure the validity of the computed results. Nodal displacements and stresses due to the individual loads are the most appropriate data for this purpose. In particular, displacements and stresses across the upper arch and the crown cantilever sections are extremely helpful. Such data are inspected for any unusual distributions and magnitudes that cannot be explained by intuition and which differ significantly from the results for similar arch dams. Once the accuracy of the analytical results has been accepted, the performance of the dam for the postulated loading combinations must be evaluated.

b. This second stage of evaluation involves comparing the maximum calculated stresses with the specified strength of the concrete according to the criteria established in Chapter 11. The analysis should include the effects of all actual static loads that will act on the structure during the operations, in accordance with the "Load Combinations" criteria presented in Chapter 4. The largest compressive and tensile stress for each load combination case should be less than the compressive and tensile strength of the concrete by the factors of safety specified for each design load combination. When design criteria for all postulated loads are met and the factors of safety are in the acceptable range, the design is considered satisfactory, or, in the case of an existing dam, it is considered safe under the static loads. However, if calculated tensile stresses exceed the cracking strength of the concrete or the lift joints or if tensile stresses are indicated across the vertical monolith joints, the possibility of tension cracking and joint opening must be considered and judgement is required to interpret the results.

(1) Under the static loads, a well designed arch dam should develop essentially compressive stresses that are significantly less than the compressive strength of the concrete; however, tensile stresses may be developed under multiple loading combinations, particularly when the temperature drop is large and other conditions are unfavorable. Although unreinforced concrete can tolerate a limited amount of tensile stress, it is important to keep the

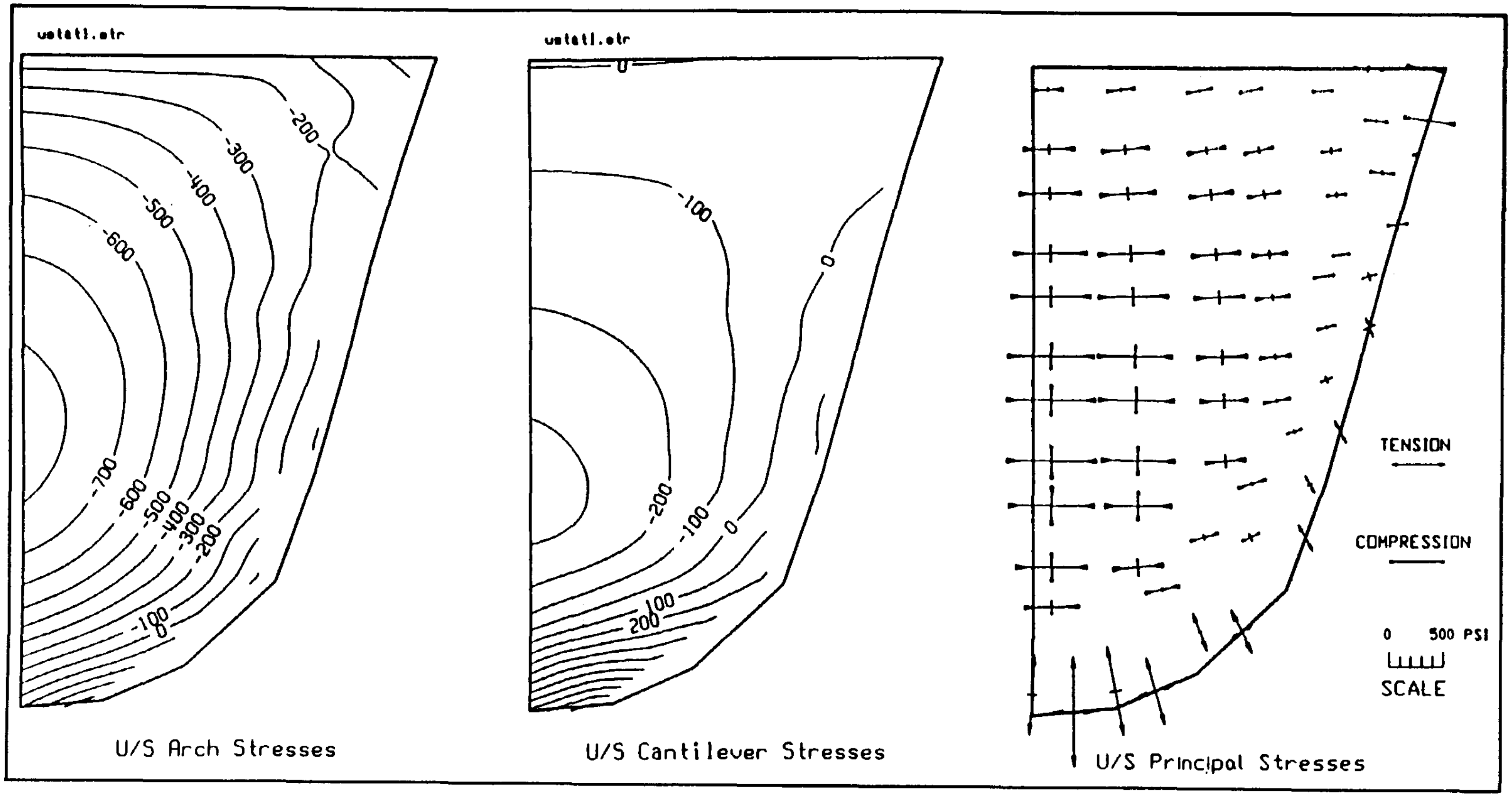


Figure 6-9. Arch and cantilever stress contours and vector plot of principal stresses for upstream face of Morrow Point Dam

tension to a minimum so that the arch has sufficient reserve strength if subjected to additional seismic loads. Vertical (cantilever) tensile stresses can be minimized by vertical arching and overhanging of the crest, but the amount by which this can be done is limited by the stress and stability of individual cantilever blocks during the construction process. When the design limits are reached or, as in the case of many existing dams, when the dam is not designed for severe loading conditions, some cracking could occur at the base and near the abutments. Linear-elastic analyses often indicate large stresses near the geometric discontinuity at the foundation contact. However, it is important to note that the tensile stresses indicated at the base of the arch dams by linear-elastic analyses are partly fictitious because these analyses do not take into account the limited bond between the concrete and foundation rock as well as the joints in the rock that could open when subjected to tensile forces. In this situation, a more realistic estimate of static stresses at the base of the dam may be obtained by a linear-elastic analysis that uses a reduced foundation deformation modulus to decrease the tension in the fractured rock.

(2) Arch dams rely significantly on arch action to transfer horizontal loads to the foundation. Therefore, in general, compressive arch stresses are expected throughout the dam; however, the analyses of monolithic arch dams with empty reservoirs, with low water levels, or with severe low temperatures have indicated that zones of horizontal tensile stresses can develop on the upstream and downstream dam faces. These tensile stresses combined with additional tensile stresses due to temperature drop tend to open the vertical contraction joints which are expected to have little or no tensile strength. It is apparent that joint opening will relieve any indicated arch tensile stresses, and the corresponding loads can be redistributed to cantilever action provided that tensile arch stresses are limited to only a small portion of the dam.

(3) Shear stresses are rarely a problem in an arch dam; nevertheless, they should be checked to make sure that they remain within the allowable limits.

c. In conclusion, the results of a linear elastic analysis are valid only if the cracking or joint openings that occur in the dam are minor and the total stiffness of the structure is not affected significantly. Therefore, it is necessary to evaluate the extent of cracking and to judge whether or not a state of no tension can safely be achieved in the dam and its foundation. If appreciable cracking is indicated, it is desirable to investigate its extent and its effects on actual stresses and deflections by analytical procedures. An approximate investigation based on a simplified nonlinear analysis may be made by eliminating the tension areas by iteration and reanalyzing the arch.

CHAPTER 7

EARTHQUAKE RESPONSE ANALYSIS

7-1. Introduction. A dynamic method of analysis is required to properly assess the safety of existing concrete arch dams and to evaluate proposed designs for new dams that are located in regions with significant seismicity. Dynamic analysis is also performed to determine the adequacy of structural modifications proposed to improve the seismic performance of old dams. The prediction of the actual dynamic response of arch dams to earthquake loadings is a very complicated problem and depends on several factors including intensity and characteristics of the design earthquakes, interaction of the dam with the foundation rock and reservoir water, computer modeling, and the material properties used in the analysis. Detailed descriptions of the recommended dynamic analysis procedures are provided in the "Theoretical Manual for Analysis of Arch Dams" (Ghanaat 1993b). Guidance concerning the seismic studies needed to specify the design earthquake ground motions, methods of analysis, parameters influencing the dam response, and the presentation and evaluation of the analysis results are discussed in this chapter.

7-2. Geological-Seismological Investigation. Estimation of appropriate seismic excitation parameters is an important aspect of the seismic design, analysis, and evaluation of new and existing dams. Concrete arch dams built in seismic regions may be subjected to ground shaking due to an earthquake at the dam site or, more likely, to ground motions induced by distant earthquakes. In addition, large dams may experience earthquakes triggered at the dam site immediately following the reservoir impoundment or during a rapid drawdown. However, such reservoir-induced earthquakes are usually no greater than those to be expected without the reservoir, and they do not augment the seismicity of the region. The estimation of future earthquake ground motions at a dam site requires geological, seismological, geophysical, and geotechnical investigations. The primary purposes of these studies are to establish the tectonic and geologic setting at and in the vicinity of the dam site, to identify active faults and seismic sources, to collect and analyze the historic and instrumental seismic data, and to study the foundation conditions at the dam site that form the basis for estimating the ground motions. However, the lack of necessary data or difficulty in obtaining them, as well as numerous uncertainties associated with the source mechanism and the seismic wave propagation, often complicate the estimation process of ground motions. Therefore, at the present time seismic parameters for dam projects are approximated by empirical relations and through simplified procedures that decouple or neglect the effects of less understood phenomena. The primary factors that must be considered in determination of the seismic parameters for dam projects are discussed in the following paragraphs.

a. Regional Geologic Setting. A study of regional geology is required to understand the overall geologic setting and seismic history of a dam site. The study area, as a minimum, should cover a 100 km radius around the site. But in some cases it may be extended to as far as 300 km in order to include all significant geologic features such as major faults and to account for area-specific attenuation of earthquake ground motion with distance. A typical geologic study consists of:

(1) Description of the plate tectonic setting of the dam region together with an account of recent movements.

(2) Regional geologic history and physiographic features.

(3) Description of geologic formations, rock types, soil deposits.

(4) Compilation of active faults in the site region and assessment of the capability of faults to generate earthquakes.

(5) Characterization of each capable fault in terms of its maximum expected earthquake, recurrence intervals, total fault length, slip rate, slip history, and displacement per event, etc. Field work such as an exploratory trench or bulldozer cuts may also be required to evaluate the seismic history.

b. Regional Seismicity. The seismic history of a region provides information on the occurrence of past earthquakes that help to identify seismicity patterns and, thus, give an indication of what might be expected in the future. Procedures for estimating the ground motion parameters at a particular site are primarily based on historic and instrumentally recorded earthquakes and other pertinent geologic considerations. It is important, therefore, to carefully examine such information for accuracy, completeness and consistency. When possible, the following investigations may be required:

(1) Identification of seismic sources significant to the site, usually within about a 200-km radius.

(2) Development of a catalog of the historical and instrumentally recorded earthquakes for the dam site region. The data, whenever possible, should include locations, magnitudes or epicentral intensity, date and time of occurrence, focal depth, and focal mechanism.

(3) Illustration of the compiled information by means of appropriate regional and local seismicity maps.

(4) Analysis of seismicity data to construct recurrence curves of the frequency of earthquakes for the dam region, to examine spatial patterns of epicenters for possible connection with the identified geologic structures, and to evaluate the catalog for completeness and accuracy.

(5) A review of the likelihood of reservoir induced seismicity (RIS) at the dam site, although this is not expected to influence the design earthquake parameters as previously mentioned.

c. Local Geologic Setting. Local geology should be studied to evaluate some of the site-specific characteristics of the ground motion at the dam site. Such data include rock types, surface structures, local faults, shears and joints, and the orientation and spacing of joint systems. In some cases, there may be geologic evidence of primary or sympathetic fault movement through the dam foundation. In those situations, a detailed geologic mapping, and geophysical and geotechnical exploration should be carried out to assess the potential, amount, and the type of such movements at the dam site.

7-3. Design Earthquakes. The geological and seismological investigations described in the previous paragraph provide the basis for estimating the earthquake ground motions to be used in the design and analysis of arch dams. The level of such earthquake ground motions depend on the seismic activity in the dam site vicinity, source-to-site distance, length of potential fault ruptures, source mechanism, surface geology of the dam site, and so on. Two approaches are available for estimating the ground motion parameters: deterministic and probabilistic. Both approaches require specification of seismic sources, assessing maximum magnitudes for each of the sources, and selecting ground motion attenuation relationships. The probabilistic analysis requires the additional specification of the frequency of earthquake recurrence for each of the sources in order to evaluate the likelihood of exceeding various level of ground motion at the site. The earthquake ground motions for which arch dams should be designed or analyzed include OBEs and MDEs. The ground motions defined for each of these earthquakes are discussed in the following paragraphs.

a. Operating Basis Earthquake (OBE). The OBE is defined as the ground motion with a 50 percent probability of being exceeded in 100 years. In design and safety evaluation of arch dams, an OBE event should be considered as an unusual loading condition as described in Chapter 4. The dam, its appurtenant structures, and equipment should remain fully operational with minor or no damage when subjected to earthquake ground motions not exceeding the OBE.

b. Maximum Design Earthquake (MDE). The MDE is the maximum level of ground motion for which the arch dam should be analyzed. The MDE is usually equated to the MCE which, by definition, is the largest reasonably possible earthquake that could occur along a recognized fault or within a particular seismic source zone. In cases where the dam failure poses no danger to life or would not have severe economic consequences, an MDE less than the MCE may be used for economic reasons. An MDE event should be considered as an extreme loading condition for which significant damage is acceptable, but without a catastrophic failure causing loss of life or severe economic loss.

c. Reservoir-induced Earthquake (RIE). The reservoir-induced earthquake is the maximum level of ground motion that may be triggered at the dam site during filling, rapid drawdown, or immediately following the reservoir impoundment. Statistical analysis of the presumed RIE cases have indicated a relation between the occurrence of RIE and the maximum water depth, reservoir volume, stress regime, and local geology. The likelihood of an RIE is normally considered for dams higher than about 250 feet and reservoirs with capacity larger than about 10^5 acre-feet, but the possibility of an RIE occurring at new smaller dams located in tectonically sensitive areas should not be ruled out. The possibility of RIE's should therefore be considered when designing new high dams, even if the region shows low historical seismicity. The determination of whether the RIE should be considered as a dynamic unusual or a dynamic extreme loading condition (Table 4-2) should be based on the probability of occurrence but recognizing that the RIE is no greater than the expected earthquake if the reservoir had not been built.

7-4. Earthquake Ground Motions. The earthquake ground motions are characterized in terms of peak ground acceleration, velocity, or displacement values, and seismic response spectra or acceleration time histories. For the evaluation of arch dams, the response spectrum and/or time-history representation of earthquake ground motions should be used. The ground motion parameters for the OBE are determined based on the probabilistic method. For the MCE, however, they are normally estimated by deterministic analysis, but a probabilistic analysis should also be considered so that the likelihood of a given intensity of ground motion during the design life of the dam structure can be determined. The earthquake ground motions required as input for the seismic analysis of arch dams are described in the following subparagraphs.

a. Design Response Spectra. The ground motion used for the seismic analysis of arch dams generally is defined in the form of smooth response spectra and the associated acceleration time histories. In most cases site-specific response spectra are required, except when the seismic hazard is very low; in which case a generic spectral shape such as that provided in most building codes may suffice. When site-specific response spectra are required, the effects of magnitude, distance, and local geological conditions on the amplitude and frequency content of the ground motions should be considered. In general, the shape of the response spectrum for an OBE event is different from that for the MCE, due to differences in the magnitude and the earthquake sources as shown in Figure 7-1. Thus, two separate sets of smooth response spectra may be required, one for the OBE and another for the MCE. The smooth response spectra for each design earthquake should be developed for both horizontal and vertical components of the ground motion. The design spectra are typically developed for 5 percent damping. Estimates for other damping values can be obtained using available relationships (Newmark and Hall 1982). The vertical response spectra can be estimated using the simplified published relationships between the vertical and horizontal spectra which will be described in a future engineer manual. The relationship used should recognize the significant influence of the source-to-site distance and of the particular period range (≤ 0.2 sec) on the vertical response spectra.

b. Acceleration Time Histories. When acceleration time histories of ground motions are used as seismic input for the dynamic analysis of arch dams, they should be established with the design response spectra and should have appropriate strong motion duration and number of peaks. The duration of strong motion is commonly measured by the *bracketed duration.* This is the duration of shaking between the first and last accelerations of the accelerogram exceeding 0.05 g.

(1) Acceleration time histories are either selected from recorded ground motions appropriate to the site, or they are synthetically developed or modified from one or more ground motions. In the first approach, several records are usually required to ensure that the response spectra of all records as a whole do not fall below the smooth design spectra. This procedure has the advantage that the dam is analyzed for natural motions, several dynamic analyses should be performed. In addition, the response spectrum of individual records may have peaks that substantially exceed the design response spectra.

(2) Alternatively, acceleration time histories are developed either by artificially generating an accelerogram or by modifying a recorded accelerogram so that the response spectrum of the resulting accelerogram closely matches the design response spectra. The latter technique is preferred,

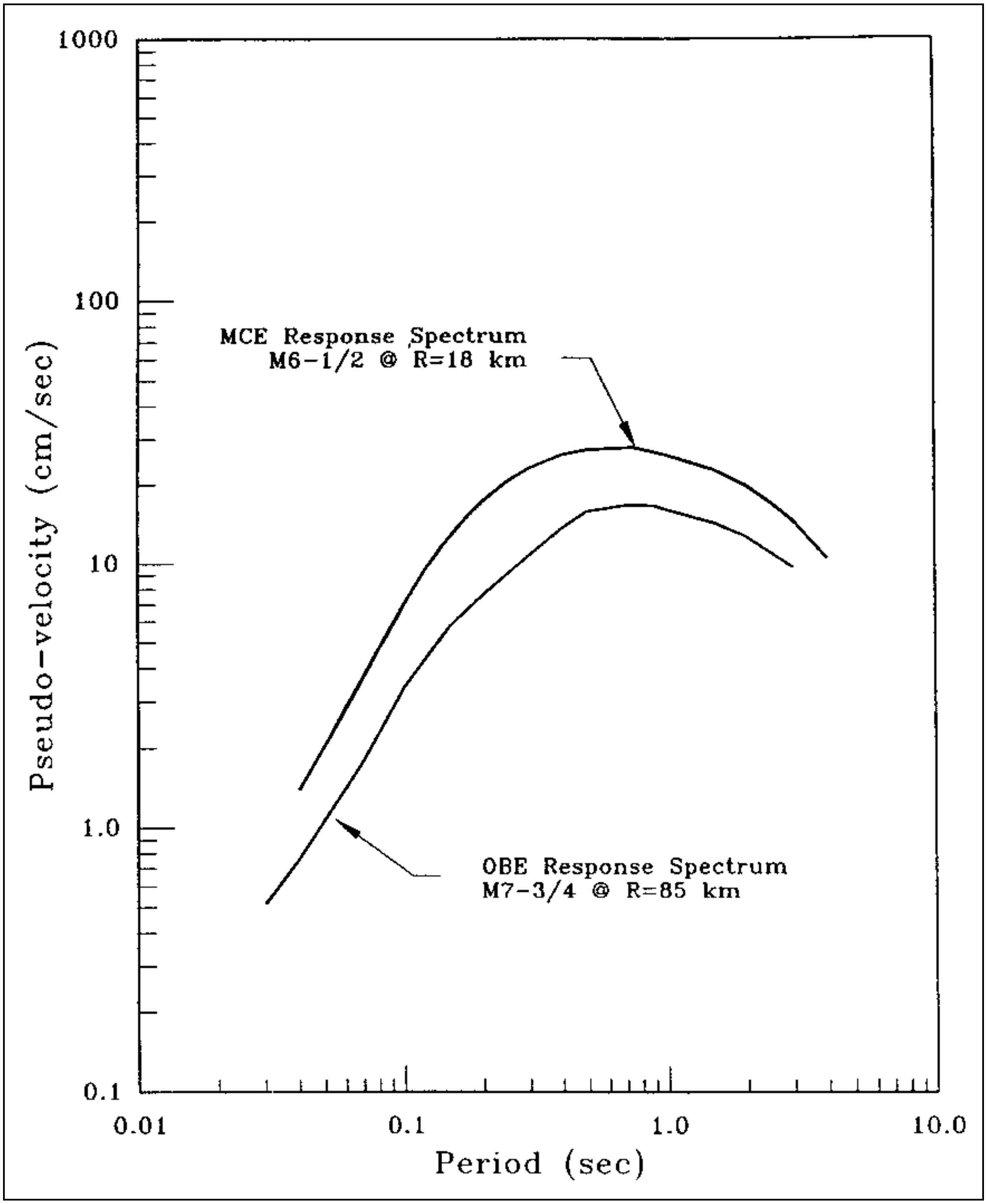


Figure 7-1. Smooth design response-spectrum examples for OBE and MCE events for 5 percent damping

because it starts with a natural accelerogram and thus preserves the duration and phasing of the original record and produces time histories that look natural. An example of this procedure which shows a good match with the smooth design response spectrum is demonstrated in Figure 7-2.

(3) For large thin arch dams with fundamental periods near 0.5 to 1 sec located at close distances to the earthquake source, it is desirable to include a strong intermediate-to-long period pulse (0.5 to 5 sec) to account for the "fling" characteristic of near-source ground motion.

7-5. Finite Element Modeling Factors Affecting Dynamic Response. Dynamic analysis of arch dams for earthquake loading should be based on a 3-D idealization of the dam-water-foundation system which accounts for the significant interaction effects of the foundation rock and the impounded water. To compute the linear response of the dam, the concrete arch and the foundation rock are modeled by standard finite elements, whereas the interaction effects of the impounded water can be represented with any of three different level of refinement. In addition, the dynamic response of arch dams is affected by the damping and by the intensity and spatial variation of the seismic input. These factors and the finite element modeling of various components of an arch dam are discussed in the following sections.

a. Arch Dam. The finite element model of an arch dam for dynamic analysis is essentially identical to that developed for the static analysis. In a linear-elastic analysis, the arch dam is modeled as a monolithic structure with no allowance for the probable contraction joint opening during earthquake excitation. Thin and moderately thin arch dams are adequately modeled by a single layer of shell elements, whereas thick gravity-arch dams should be represented by two or more layers of solid elements through the dam thickness. The size of the mesh should be selected following the general guidelines presented in Chapter 6 for static analysis and shown in Figure 6-1. In addition, the dynamic response of the appurtenant structures attached to the dam may be significant and also should be considered. For example, the power intakes attached to the dam may include free-standing cantilevers that could vibrate during the earthquake shaking. The power intakes in this case should be included as part of the dam model to ensure that the dam stresses induced by the vibration of these components are not excessive.

b. Dam-foundation Rock Interaction. Arch dams are designed to resist the major part of the water pressures and other loads by transmitting them through arch action to the canyon walls. Consequently, the effects of foundation rock on the earthquake response of arch dams are expected to be significant and must be considered in the dynamic analysis. However, a complete solution of the dam foundation interaction effects is very complicated and such procedures have not yet been fully developed. There are two major factors contributing to this complex interaction problem. First is the lack of a 3-D model of the unbounded foundation rock region to account for energy loss due to the radiation of vibration waves. The other and even more important contributing factor is related to the prescription of spatial variation of the seismic input at the dam-foundation interface, resulting from wave propagation of seismic waves through the foundation rock and from scattering by the canyon topography. Faced with these difficulties, an overly simplified model of the foundation rock (Clough 1980) is currently used in practice. This widely used simplified model ignores inertial and damping effects and considers only the

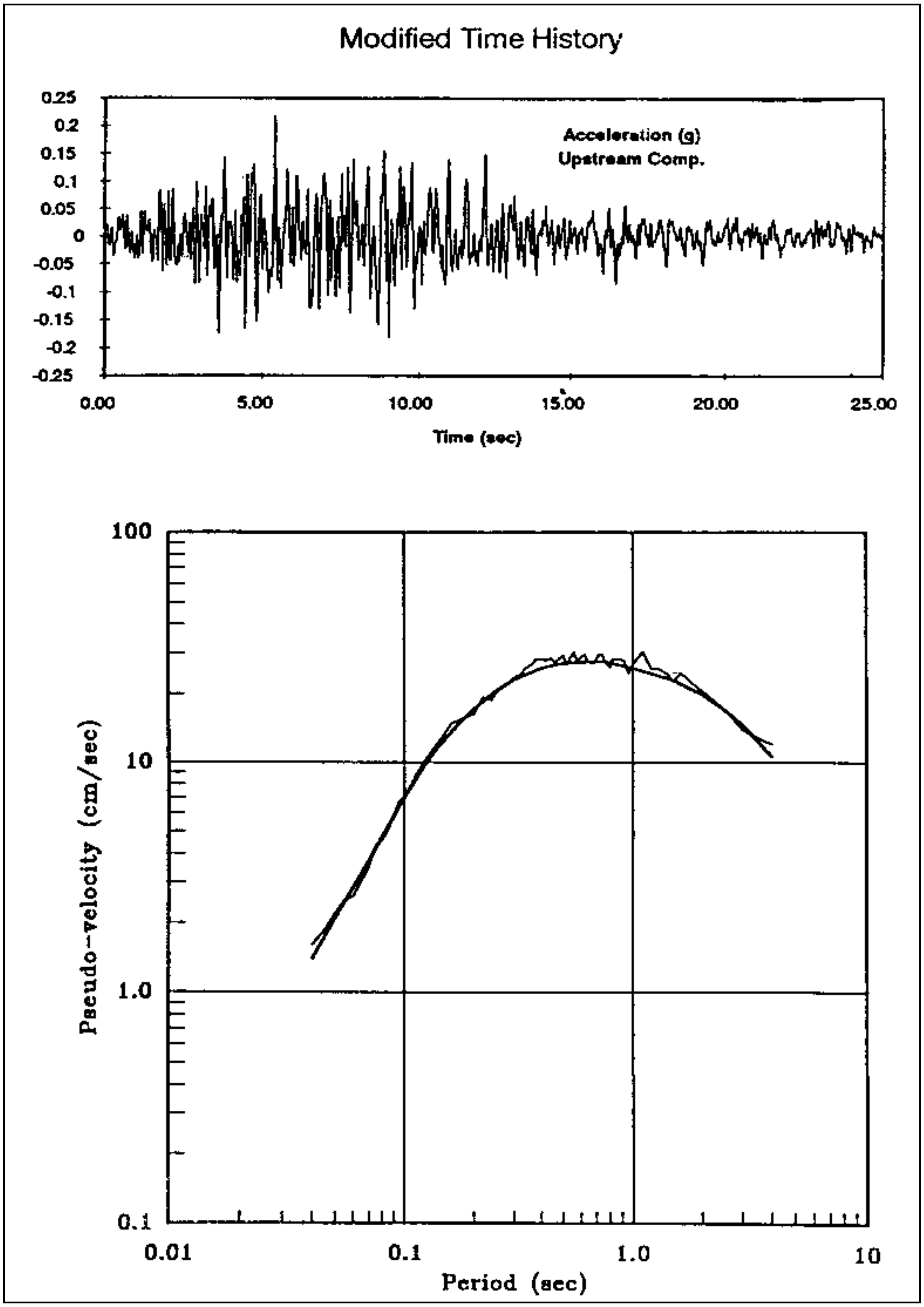


Figure 7-2. Comparison of response spectrum of modified time history and smooth-design response spectrum for 5 percent damping

flexibility of the foundation rock. The foundation model for the dynamic analysis is therefore similar to that described for the static analysis in Chapter 6. As shown in Figures 6-1a and 6-3, an appropriate volume of the foundation rock should be idealized by the finite element discretization of the rock region. Each foundation element is represented by a solid element having eight or more nodes and characterized by its dynamic deformation modulus and Poisson's ratio.

(1) Shape of Foundation Model. Using the finite element procedure, a foundation model can be developed to match the natural topography of the foundation rock region. However, such a refined model is usually not required in practice. Instead, a prismatic model employed in the GDAP program and described in Chapter 6 may be used. This foundation model depicted in Figure 6-1a is constructed on semicircular planes cut into the canyon walls normal to the dam-foundation contact surface; in moving from the base to the dam crest, each semicircle is rotated about a diameter always oriented in the upstream-downstream direction.

(2) Size of Foundation Model. The size of the foundation model considered in GDAP is controlled by the radius (R_f) of the semicircular planes described in the previous paragraph. In the static analysis discussed previously, R_f was selected so that the static displacements and stresses induced in the dam were not changed by further increase of the foundation size. In the dynamic analysis, the natural frequencies and mode shapes of vibration control the dam response to earthquakes. Therefore, the size of a foundation model should be selected so that the static displacements and stresses, as well as the natural frequencies and mode shapes, are accurately computed. The natural frequencies of the dam-foundation system decrease as the size of the flexible foundation rock increases (Clough et al. 1985 and Fok and Chopra 1985), but for the massless foundation, the changes are negligible when the foundation size R_f is greater than one dam height, except for the foundation rocks with very low modulus of elasticity. For most practical purposes, a massless foundation model with R_f equal to one dam height is adequate. However, when the modulus ratio of the rock to concrete is less than one-half, a model with R_f equal to two times dam height should be used.

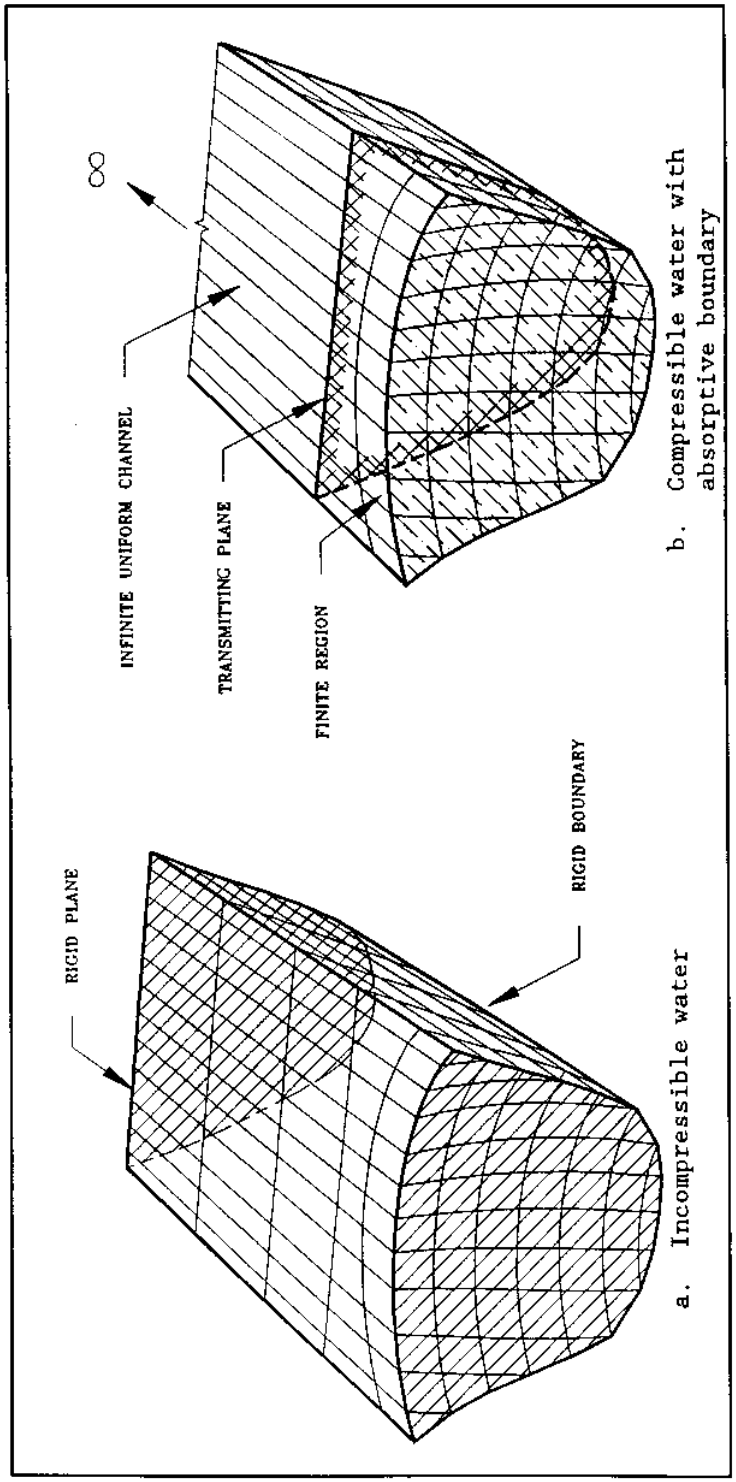
c. Dam-water Interaction. Interaction between the dam and impounded water is an important factor affecting the dynamic response of arch dams during earthquake ground shaking. In the simplest form, this interaction can be represented by an "added mass" attached to the dam first formulated by Westergaard (1933). A more accurate representation of the added mass is obtained using a finite element formulation which accounts for the complicated geometry of the arch dam and the reservoir (Kuo 1982 (Aug)). Both approaches, however, ignore compressibility of water and the energy loss due to radiation of pressure waves in the upstream direction and due to reflection and refraction at the reservoir bottom. These factors have been included in a recent and more refined formulation (Fok and Chopra 1985 (July)), but computation of the resulting frequency-dependent hydrodynamic pressure terms requires extensive efforts and requires consideration of a range of reservoir-bottom reflection coefficients.

(1) Generalized Westergaard Added Mass. Westergaard (1933) demonstrated that the effects of hydrodynamic pressures acting on the vertical face of a rigid gravity dam could be represented by an added mass attached to the

dam, if the compressibility of water is neglected. A general form of this incompressible added-mass concept has been applied to the analysis of arch dams (Kuo 1982 (Aug)). This generalized formulation, also described by Ghanaat (1993b), is based on the same parabolic pressure distribution in the vertical direction used by Westergaard, but it recognizes the fact that the hydrodynamic pressures acting on the curved surface of an arch dam are due to the total accelerations normal to the dam face. Although the resulting added mass calculated in this manner is often used in the analysis of arch dams, it does not properly consider the hydrodynamic effects. In fact, there is no rational basis for the assumed parabolic pressure distribution used for the arch dams, because limitations imposed in the original Westergaard formulation are violated. The original Westergaard formulation assumed a rigid dam with a vertical upstream face and an infinite reservoir. However, the procedure is very simple and provides a reasonable estimate of the hydrodynamic effects for preliminary or feasibility analysis. The generalized added-mass formulation has been implemented in the GDAP program and is available as an option. The program automatically calculates the added mass for each nodal point on the upstream face of the dam; the resulting added mass of water is then added to the mass of concrete to account for the hydrodynamic forces acting on the dam.

(2) Incompressible Finite Element Added Mass. In more refined analyses of new and existing arch dams, the effects of reservoir-water interaction due to seismic loading is represented by an equivalent added mass of water obtained from the hydrodynamic pressures acting on the face of the dam. The procedure is based on a finite element solution of the pressure wave equation subjected to appropriate boundary conditions (Kuo 1982 (Aug) and Ghanaat (1993b)). The nodal point pressures of the incompressible water elements are the unknowns. The bottom and sides of the reservoir, as well as a vertical plane at the upstream end, are assumed to be rigid. In addition, the hydrodynamic pressures at the water-free surface are set to zero; thus the effects of surface waves are neglected, but these have little effect on the seismic response. In general, a finite element model of the reservoir water can be developed to match the natural canyon topography, but a prismatic reservoir model available in the GDAP program is quite adequate in most practical situations. The GDAP reservoir model is represented by a cylindrical surface generated by translating the dam-water interface nodes in the upstream direction as shown in Figure 7-3a. The resulting water nodes generated in this manner match those on the dam face and are usually arranged in successive planes parallel to the dam axis, with the distance between the planes increasing with distance from the dam. Experience shows that the reservoir water should include at least three layers of elements that extend upstream a distance at least three times the water depth.

(3) Compressible Water with Absorptive Reservoir Bottom. The added-mass representation of hydrodynamic effects ignores both water compressibility effects and the energy absorption mechanism at the reservoir bottom. These factors have been included in a recent formulation of the dam-water interaction mechanism which is fully described by Fok and Chopra (1985 (July)). It introduces frequency-dependent hydrodynamic terms in the equations of motion that can be interpreted as an added mass, an added damping, and an added force. The added damping term arises from the refraction of hydrodynamic pressure waves into the absorptive reservoir bottom and also from the propagation of pressure waves in the upstream direction. The energy loss at the reservoir bottom is approximated by the wave reflection coefficient α, which



a. Incompressible water

b. Compressible water with absorptive boundary

Figure 7-3. Finite element models of fluid domain with and without water compressibility is defined as the ratio of reflected-to-incident wave amplitude of a pressure wave striking the reservoir bottom. The values of α can be varied from $\alpha = 1.0$, for a rigid, nonabsorptive boundary similar to that used in the GDAP model, to $\alpha = 0.0$, indicating total absorption.

(a) The response analysis of an arch dam including the effects of dam-water interaction, water compressibility, and reservoir-bottom absorption can be performed using the EACD-3D program (Fok, Hall, and Chopra 1986 (July)). The finite element idealizations of the dam and foundation rock employed in this program are essentially equivalent to those employed by the GDAP program; the fluid region near the dam is modeled by liquid finite elements similar to those in GDAP, but, unlike the GDAP, these elements are of compressible water and are connected to a uniform channel extending to infinity to permit pressure waves to radiate away from the dam (Figure 7-3b).

(b) Another major difference of the EACD-3D model is that the reservoir boundary is absorptive and thus dissipation of hydrodynamic pressure waves in the reservoir bottom materials is permitted. However, this method requires considerable computational effort and is too complicated for most practical applications. An even more important consideration is the lack of guidance or measured data for determining an appropriate α factor for use in the analysis. Consequently, such analyses must be repeated for a range of α factors in order to establish a lower and upper bound estimate of the dam response. It is also important to note that the significance of water compressibility depends on the dynamic characteristics of the dam and the impounded water. Similar to gravity dams (Chopra 1968), the effects of water compressibility for an arch dam can be neglected if the ratio of the natural frequency of the reservoir water to the natural frequency of the arch dam-foundation system without water is greater than 2.

d. Damping. Damping has a significant effect on the response of an arch dam to earthquake and other dynamic loads. The energy loss arises from several sources including the concrete arch structure, foundation rock, and the reservoir water. Dissipation of energy in the concrete arch structure is due to internal friction within the concrete material and at construction joints. In the foundation rock this energy loss is facilitated by propagation of elastic waves away from the dam (radiation damping) and by hysteretic losses due to sliding on cracks and fissures within the rock volume. An additional source of damping, as discussed in paragraph 7-5c(3), is associated with the energy loss due to refraction of hydrodynamic pressure waves into the reservoir bottom materials and propagation of pressure waves in the upstream direction.

(1) The current standard earthquake analysis of arch dams is based on a massless foundation rock model and employs incompressible added mass for representing the hydrodynamic effects. In this type of analysis, only the material damping associated with the concrete structure is explicitly considered. The overall damping constant for the entire model in such linear-elastic analyses is normally specified based on the amplitude of the displacements, the opening of the vertical contraction joints, and the amount of cracking that may occur in the concrete arch. Considering that the measured damping values for concrete dams subjected to earthquake loading are scarce and that the effects of contraction joints, lift surfaces, and cracks cannot

be precisely determined, the damping value for a moderate shaking such as an OBE event should be limited to 5 percent.

(2) However, under the MCE earthquake ground motions, damping constants of 7 or 10 percent may be used depending on the level of strains developed in the concrete and the amount of nonlinear joint opening and/or cracking that occurs. In more severe MCE conditions, especially for large dams, additional damping can be incorporated in the analysis by employing a dam-water interaction model which includes water compressibility and permits for the dissipation of energy at the reservoir boundary.

7-6. Method of Analysis. The current earthquake response analysis of arch dams is based on linear-elastic dynamic analysis using the finite element procedures. It is assumed that the concrete dam and the interaction mechanisms with the foundation rock and the impounded water exhibit linear-elastic behavior. Using this method, the arch dam and the foundation rock are treated as 3-D systems idealized by the finite element discretization discussed in previous paragraphs and in Chapter 6. Under the incompressible added-mass assumption for the impounded water, the response analysis is performed using the response-spectrum modal-superposition or the time-history method. For the case of compressible water, however, the response of the dam to dynamic loads must be evaluated using a frequency-domain procedure, in order to deal with the frequency-dependent hydrodynamic terms. These methods of analyses are discussed in the following paragraphs.

a. Response-spectrum Analysis. The response-spectrum method of analysis uses a response-spectrum representation of the seismic input motions to compute the maximum response of an arch dam to earthquake loads. This approximate method provides an efficient procedure for the preliminary analyses of new and existing arch dams. It may also be used for the final analyses, if the calculated maximum stress values are sufficiently less than the allowable stresses of the concrete. Using this procedure, the maximum response of the arch dam is obtained by combining the maximum responses for each mode of vibration computed separately.

(1) A complete description of the method is given in the theoretical manual by Ghanaat (1993b). First, the natural frequencies and mode shapes of undamped free vibration for the combined dam-water-foundation system are evaluated; the free vibration equations of motion are assembled considering the mass of the dam-water system and the stiffness of the combined dam and foundation rock models. The maximum response in each mode of vibration is then obtained from the specified response spectrum for each component of the ground motion, using the modal damping and the natural period of vibration for each particular mode. The same damping constant is used in all modes as represented by the response- spectrum curves. Since each mode reaches its maximum response at a different time, the total maximum response quantities for the dam, such as the nodal displacements and the element stresses, are approximated by combining the modal responses using the square root of the sum of the squares (SRSS) or complete quatratic combination (CQC) procedure. Finally, the resulting total maximum responses evaluated independently for each component of the earthquake ground motion are further combined by the SRSS method for the three earthquake input components, two horizontal and one vertical.

(2) For a linear-elastic response, only a few lower modes of vibration are needed to express the essential dynamic behavior of the dam structure. The appropriate number of vibration modes required in a particular analysis depends on the dynamic characteristics of the dam structure and on the nature of earthquake ground motion. But, in all cases, a sufficient number of modes should be included so that at least 90 percent of the "exact" dynamic response is achieved. Since the "exact" response values are not known, a trial-and-error procedure may be adapted, or it may be demonstrated that the participating effective modal masses are at least 90 percent of the total mass of the structure.

b. Time-history Analysis. Time-history analysis should be performed when the maximum stress values computed by the response-spectrum method are approaching or exceeding the tensile strength of the concrete. In these situations, linear-elastic time-history analyses are performed to estimate the maximum stresses more accurately as well as to account for the time-dependent nature of the dynamic response. Time-history analyses provide not only the maximum stress values, but also the simultaneous, spatial extent and number of excursions beyond any specified stress value. Thus, they can indicate if the calculated stresses beyond the allowable values are isolated incidents or if they occur repeatedly and over a significant area.

(1) The seismic input in time-history analyses is represented by the acceleration time histories of the earthquake ground motion. Three acceleration records corresponding to three components of the specified earthquake are required; they should be applied at the fixed boundaries of the foundation model in the channel, across the channel, and in the vertical directions. The acceleration time histories are established following the procedures described in paragraph 7-4b.

(2) The structural models of the dam, foundation rock, and the impounded water for a time-history analysis are identical to those developed for response-spectrum analysis. However, the solution to the equations of motion is obtained by a step-by-step numerical integration procedure. Two methods of solution are available: direct integration and mode superposition (Ghanaat, technical report in preparation). In the direct method, step-by-step integration is applied to the original equations of motion with no transformation being carried out to uncouple them. Hence, this method requires that the damping matrix to be represented is in explicit form. In practice, this is accomplished using Raleigh damping (Clough and Penzien 1975), which is of the form

$$c = a_0\, m + a_1\, k$$

where coefficients a_0 and a_1 are obtained from two given damping ratios associated with two frequencies of vibration. The direct integration method is most effective when the response is required for a relatively short duration. Otherwise, the mode superposition method in which the step-by-step integration is applied to the uncoupled equations of motion will be more efficient. In the mode superposition method, first the undamped vibration mode shapes and frequencies are calculated, and the equations of motion are transformed to those coordinates. Then the response history for each mode is evaluated separately at each time-step, and the calculated modal response histories are combined to obtain the total response of the dam structure. It should be noted that the damping in this case is expressed by the modal damping ratios and need not be specified in explicit form.

7-7. Evaluation and Presentation of Results. The earthquake performance of arch dams is currently evaluated using the numerical results obtained from a linear-dynamic analysis. The results of linear analysis provide a satisfactory estimate of the dynamic response to low- or moderate-intensity OBE earthquake motions for which the resulting deformations of the dam are within the linear-elastic range. In this case, the performance evaluation is based on simple stress checks in which the calculated elastic stresses are compared with the specified strength of the concrete. Under the MCE ground motions, it is possible that the calculated stresses would exceed the allowable values and that significant damage could occur. In such extreme cases, the dam should retain the impounded water without rupture, but the actual level of damage can be estimated only by a nonlinear analysis that takes account of the basic nonlinear behavior mechanisms such as the joint opening, tensile cracking, and the foundation failure. However, a complete nonlinear analysis is not currently possible, and linear analysis continues to be the primary tool for assessing the seismic performance of arch dams subjected to damaging earthquakes. Evaluation of the seismic performance for the MCE is more complicated, it requires some judgement and elaborate interpretations of the results before a reasonable estimate of the expected level of damage can be made or the possibility of collapse can be assessed.

a. Evaluation of Response-spectrum Analysis. The first step in response spectrum analysis is the calculation of vibration mode shapes and frequencies. The mode shapes and frequencies provide insight into the basic dynamic response behavior of an arch dam. They provide some advance indication of the sensitivity of the dynamic response to earthquake ground motions having various frequency contents. Figure 7-4 demonstrates a convenient way for presenting the mode shapes. In this figure the vibration modes are depicted as the plot of deflected shapes along the arch sections at various elevations. After the calculation of mode shapes and frequencies, the maximum dynamic response of the dam structure is computed. These usually include the maximum nodal displacements and element stresses. In particular, the element stresses are the primary response quantity used for the evaluation of earthquake performance of the dam.

(1) Dynamic Response. The basic results of a response-spectrum analysis include the extreme values of the nodal displacements and element stresses due to the earthquake loading. As discussed earlier, these extreme response values are obtained by combining the maximum responses developed in each mode of vibration using the SRSS or CQC combination rule. In addition, they are further combined by the SRSS method to include the effects of all three components of the earthquake ground motion. Thus, the resulting dynamic response values obtained in this manner have no sign and should be interpreted as being either positive or negative. For example the response-spectrum stress values are assumed to be either tension or compression.

(2) Total Response. The evaluation of earthquake performance of an arch dam using the response-spectrum method of analysis involves comparison of the total stresses due to both static and earthquake loads with the expected

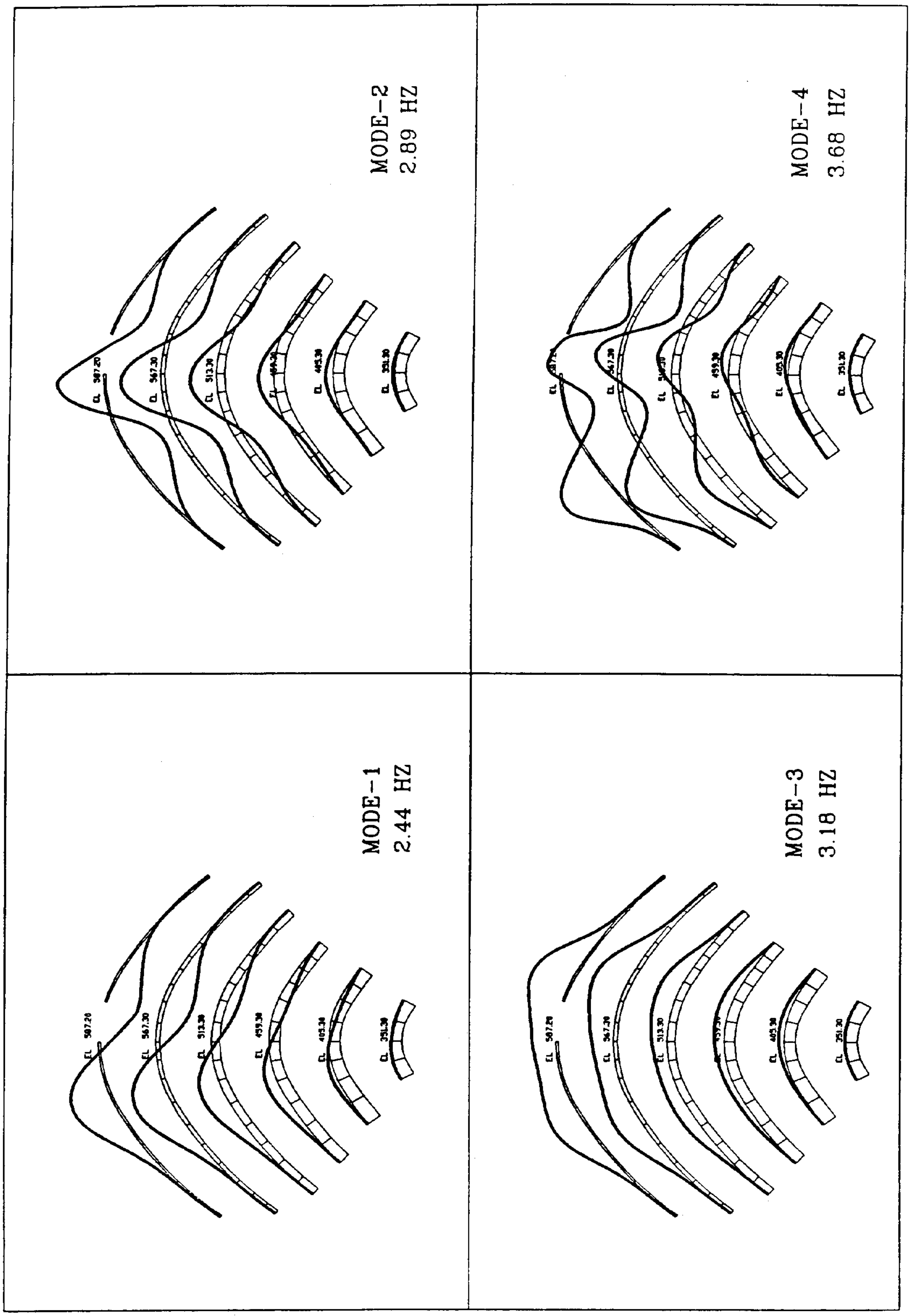


Figure 7-4. Four lowest vibration modes of Portugues Arch Dam

strength of the concrete. To obtain the total stress values, the response-spectrum estimate of the dynamic stresses (σ_d) should be combined with the effects of static loads (σ_{st}). The static stresses in a dam prior to the earthquake are computed using the procedures described in Chapter 6. The static loads to be considered include the self-weight, hydrostatic pressures, and the temperature changes that are expected during the normal operating condition as discussed in Chapter 4. Since response-spectrum stresses have no sign, combination of static and dynamic stresses should consider dynamic stresses to be either positive or negative, leading to the maximum values of total tensile or compressive stresses:

$$\sigma_{max} = \sigma_{st} \pm \sigma_d$$

(a) This combination of static and dynamic stresses is appropriate if Σ_{st} and Σ_d are oriented similarly. This is true for arch or cantilever stresses at any point on the dam surface, but generally is not true for the principal stresses. In fact, it is not possible to calculate the principal stresses from a response-spectrum analysis, because the maximum arch and cantilever stresses do not occur at the same time; therefore, they cannot be used in the principal stress formulas.

(b) The computed total arch and cantilever stresses for the upstream and downstream faces of the dam should be displayed in the form of stress contours as shown in Figure 7-5. These represent the envelopes of maximum total arch and cantilever stresses on the faces of the dam, but because they are not concurrent they cannot be combined to obtain envelopes of principal stresses, as was mentioned previously.

b. Results of Time-history Analysis. Time-history analysis computes time-dependent dynamic response of the dam model for the entire duration of the earthquake excitation. The results of such analyses provide not only the maximum response values, but also include time-dependent information that must be examined and interpreted systematically. Although evaluation of the dynamic response alone may sometimes be required, the final evaluation should be based on the total response which also includes the effects of static loads.

(1) Mode Shapes and Nodal Displacements. Vibration mode shapes and frequencies are required when the mode-superposition method of time-history analysis is employed. But it is also a good practice to compute them for the direct method. The computed vibration modes may be presented as shown in Figure 7-4 and discussed previously. The magnitude of nodal displacements and deflected shape of an arch dam provide a visual means for the evaluation of earthquake performance. As a minimum, displacement time histories for several critical nodal points should be displayed and evaluated. Figure 7-6 shows an example of such displacement histories for a nodal point on the dam crest.

(2) Envelopes of Maximum and Minimum Arch and Cantilever Stresses. Examination of the stress results for a time-history analysis should start with presentation of the maximum and minimum arch and cantilever stresses. These stresses should be displayed in the form of contour plots for the upstream and downstream faces of the dam. The contour plots of the maximum

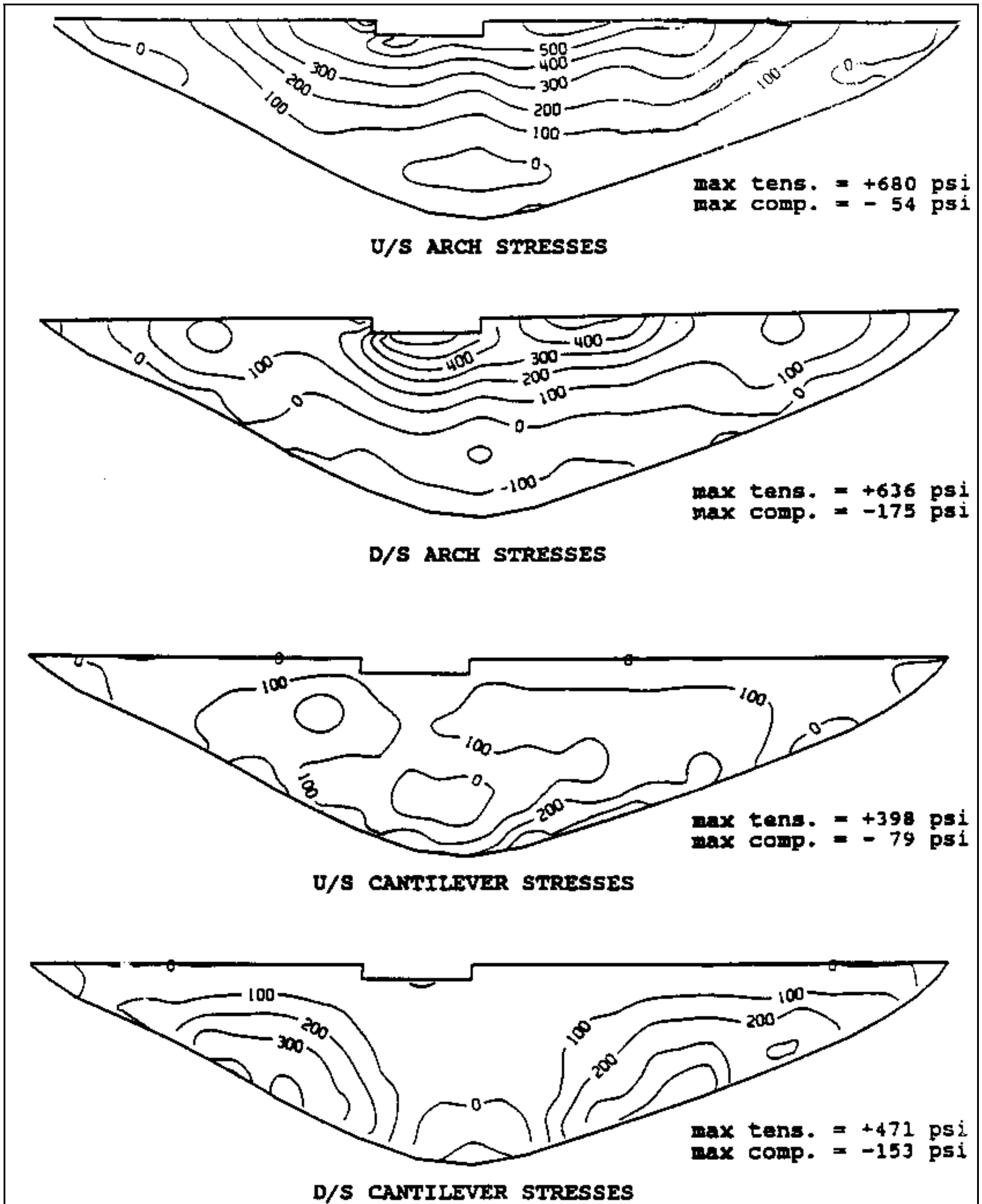


Figure 7-5. Envelope of maximum arch and cantilever stress (in psi)

arch and cantilever stresses represent the largest computed tensile (positive) stresses at all locations in the dam during the earthquake ground shaking (Figure 7-5). Similarly, the contour plots of the minimum stresses represent the largest compressive (negative) arch and cantilever stresses in the dam. The maximum and the minimum stresses at different points are generally reached at different instants of time. Contour plots of the maximum arch and cantilever stresses provide a convenient means for identifying the overstressed

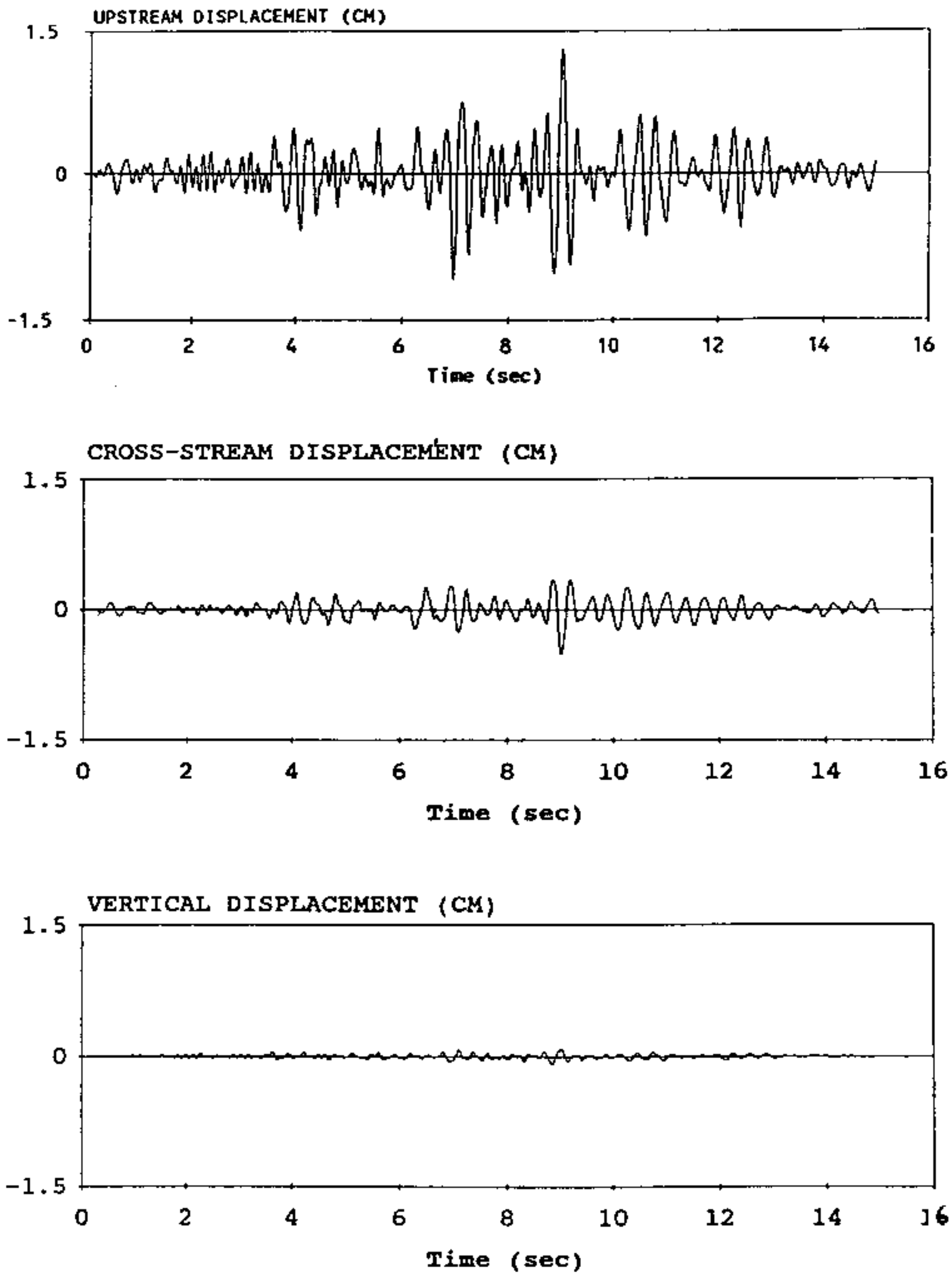


Figure 7-6. Displacement time history of a crest node in upstream, cross-stream, and vertical direction

areas where the maximum stresses approach or exceed tensile strength of the concrete. Based on this information, the extent and severity of tensile stresses are determined, and if necessary, further evaluation which accounts for the time-dependent nature of the dynamic response should be made as described in the following sections. Contour plots of the minimum stresses show the extreme compressive stresses that the dam would experience during the earthquake loading. The compressive stresses should be examined to ensure that they meet the specified safety factors for the dynamic loading (Chapter 11).

(3) Concurrent Stresses. The envelopes of maximum and minimum stresses discussed in paragraph 7-7b(2) demonstrate the largest tensile and compressive stresses that are developed at different instants of time. They serve to identify the overstressed regions and the times at which the critical stresses occur. This information is then used to produce the concurrent (or simultaneous) state of stresses corresponding to the time steps at which the critical stresses in the overstressed regions reach their maxima. The concurrent arch and cantilever stresses in the form of contour plots (Figure 7-7) can be viewed as snap shots of the worst stress conditions.

(4) Envelopes of Maximum and Minimum Principal Stresses. The time histories of principal stresses at any point on the faces of the dam are easily computed from the histories of arch, cantilever, and shear stresses at that point. When the effects of static loads are considered, the static and dynamic arch, cantilever, and shear stresses must be combined for each instant of time prior to the calculation of the total principal stresses for the same times. The resulting time histories of principal stresses are used to obtain the maxima and minima at all points on both faces of the dam which are then presented as vector plots as shown in Figures 7-8 and 7-9.

(5) Time History of Critical Stresses. When the maximum and concurrent stresses show that the computed stresses exceed the allowable value, the time histories of critical stresses should be presented for a more detailed evaluation (Figure 7-10). In this evaluation the time histories for the largest maximum arch and cantilever stresses should be examined to determine the number of cycles that the maximum stresses exceed the allowable value. This would indicate whether the excursion beyond the allowable value is an isolated case or is repeated many times during the ground motion. The total duration that the allowable value (or cracking stress) is exceeded by these excursions should also be estimated to demonstrate whether the maximum stress cycles are merely spikes or they are of longer duration and, thus, more damaging. The number of times that the allowable stress can safely be exceeded has not yet been established. In practice, however, up to five stress cycles have been permitted based on judgement but have not been substantiated by experimental data. The stress histories at each critical location should be examined for two opposite points on the upstream and downstream faces of the dam as in Figure 7-10. For example, a pair of cantilever stress histories can demonstrate if stresses on both faces are tension, or if one is tension and the other is compression. The implication of cantilever stresses being tension on both faces is that the tensile cracking may penetrate through the dam section, whereas in the case of arch stresses, this indicates a complete separation of the contraction joint at the location of maximum tensile stresses.

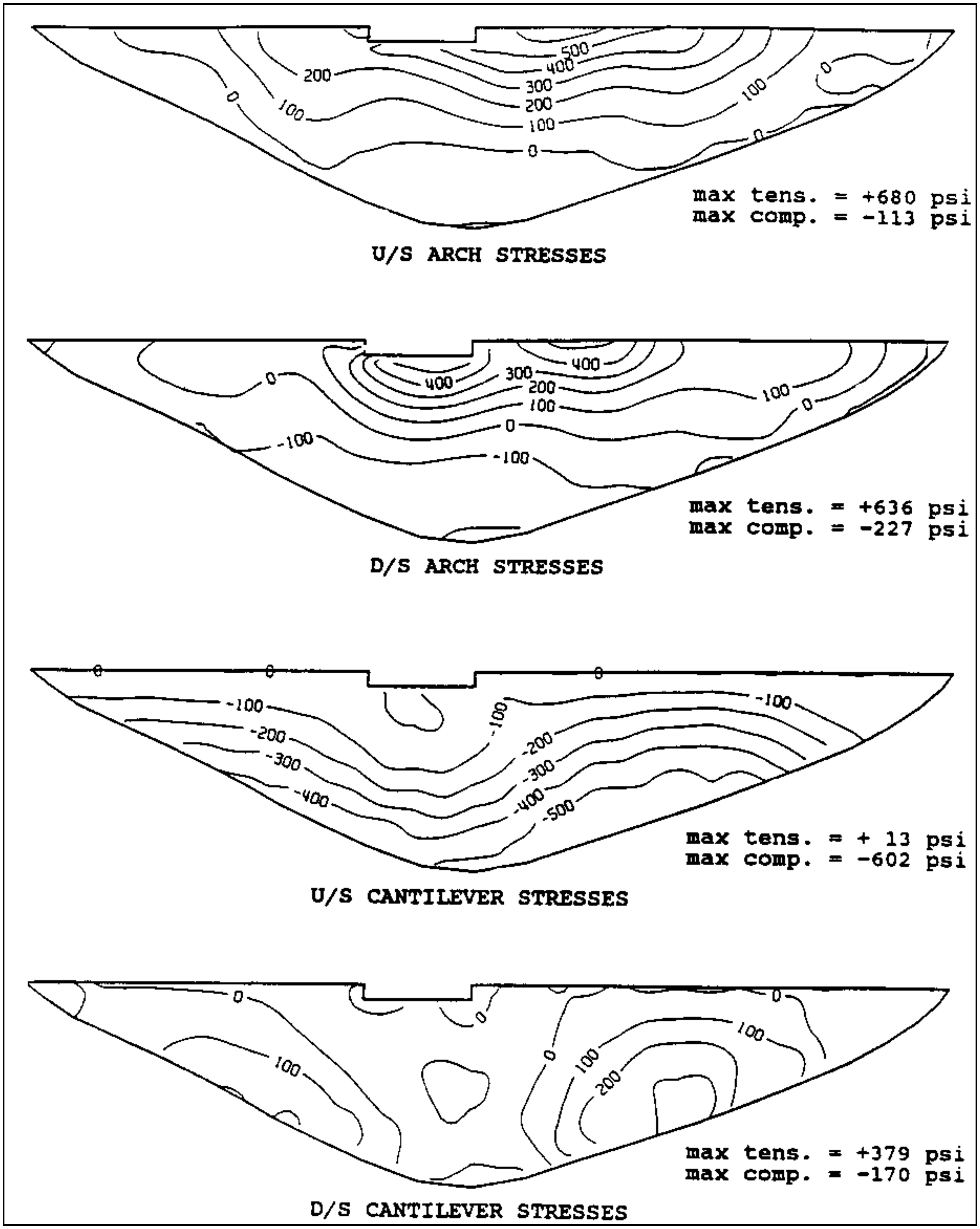


Figure 7-7. Concurrent arch and cantilever stresses (in psi) at time-step corresponding to maximum arch stress

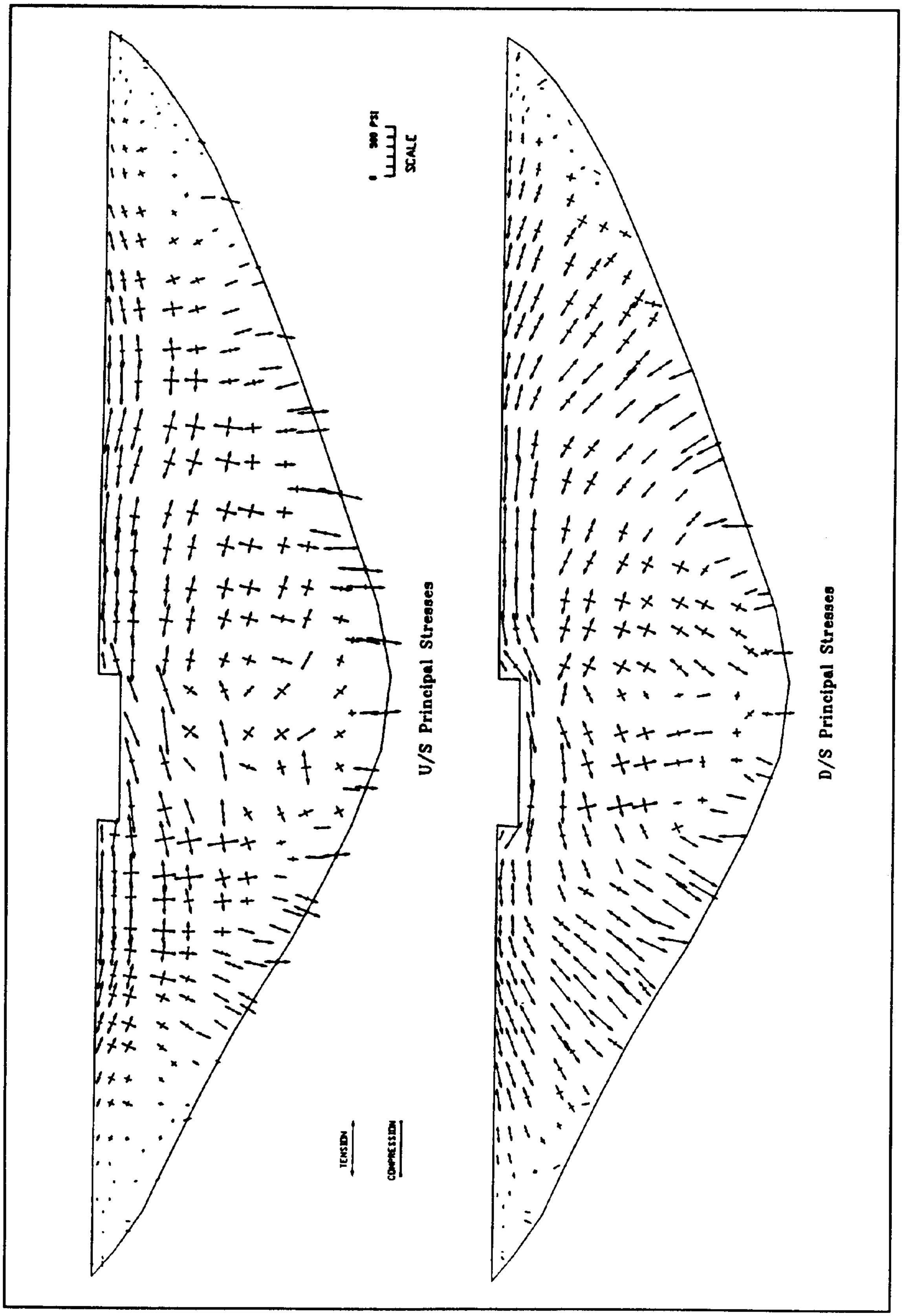


Figure 7-8. Envelope of maximum principal stresses with their corresponding perpendicular pair

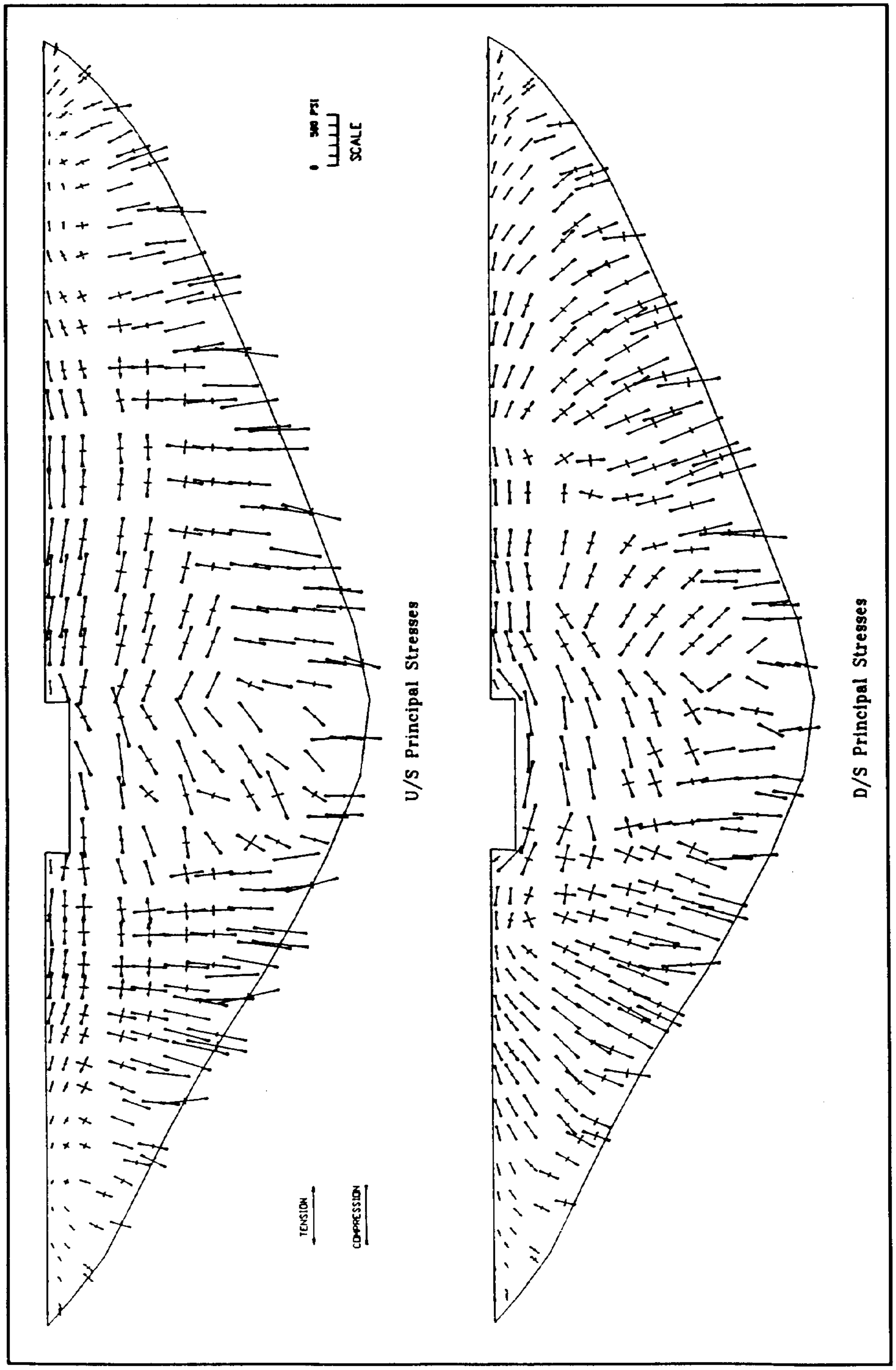


Figure 7-9. Envelope of maximum-minimum principal stresses with their corresponding perpendicular pair

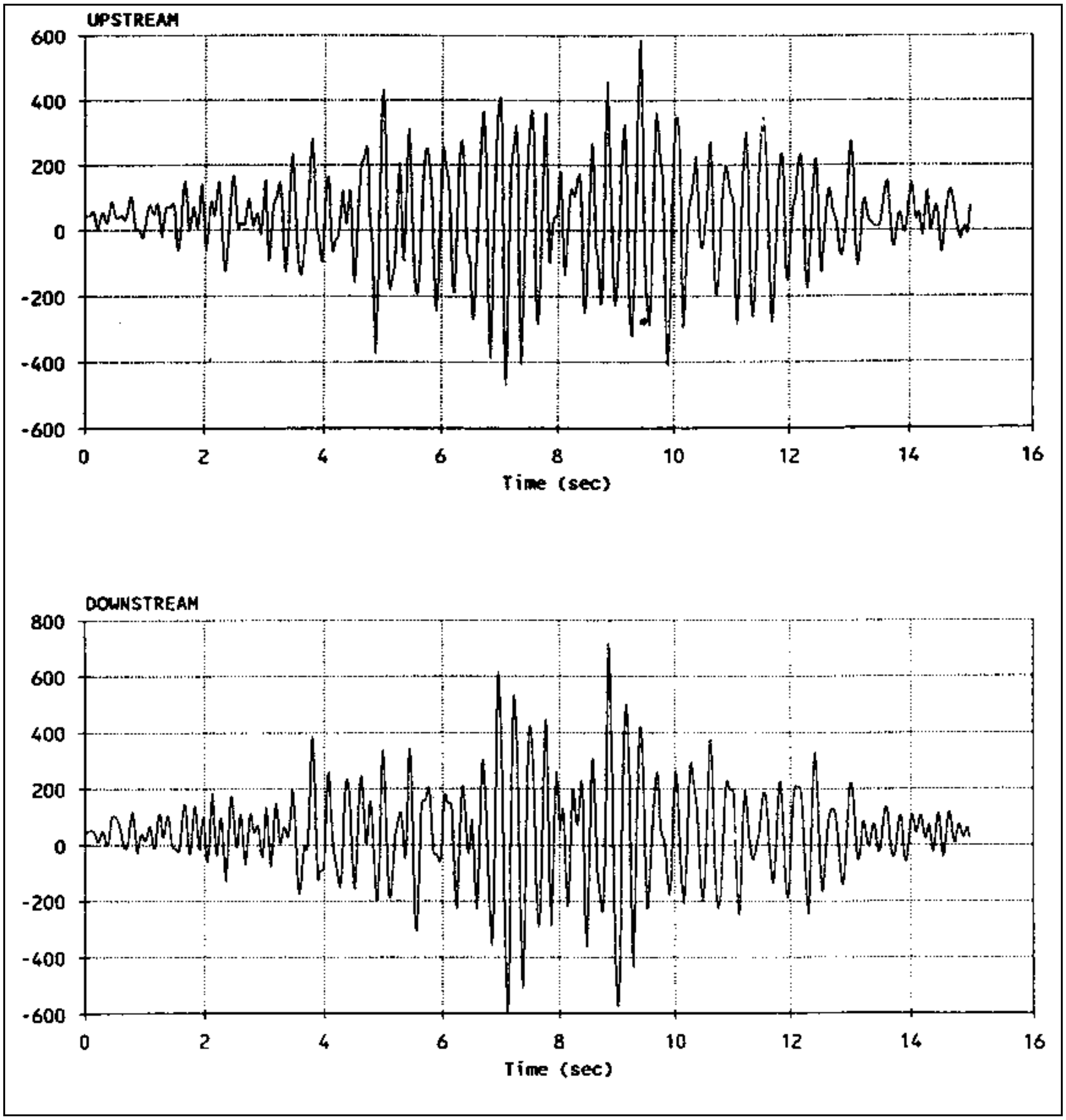


Figure 7-10. Time histories of arch stresses (in psi) at two opposite points on upstream and downstream faces of dam

CHAPTER 8

TEMPERATURE STUDIES

8-1. Introduction. Temperature studies for arch dams fall into two distinct categories. The first category is the operational temperature study which is used to determine the temperature loading in the dam. This study is performed early in the design process. The second category includes the construction temperature studies which are usually performed after an acceptable layout has been obtained. The construction temperature studies are needed to assure that the design closure temperature can be obtained while minimizing the possibility of thermally induced cracking. The details of each of these studies are discussed in this chapter. Guidance is given on when the studies should be started, values that can be assumed prior to completion of the studies, how to perform the studies, and what information is required to do the studies.

8-2. Operational Temperature Studies.

a. General. The operational temperature studies are studies that are performed to determine the temperature distributions that the dam will experience during its expected life time. The shape of the temperature distribution through the thickness of the dam is, for the most part, controlled by the thickness of the structure. Dams with relatively thin sections will tend to experience temperature distributions that approach a straight line from the reservoir temperature on the upstream face to the air temperature on the downstream face as shown in Figure 8-1. Dams with a relatively thick section will experience a somewhat different temperature distribution. The temperatures in the center of a thick section will not respond as quickly to changes as temperatures at the faces. The temperatures in the center of the section will remain at or about the closure temperature,[1] with fluctuations of small amplitude caused by varying environmental conditions. The concrete in close proximity to the faces will respond quickly to the air and water temperature changes. Therefore, temperature distributions will result that are similar to those shown in Figure 8-2.

(1) Before describing how these distributions can be obtained for analysis, a description of how the temperatures are applied in the various analysis tools is appropriate. During the early design stages, when a dam layout is being determined, the trial load method is used. The computerized version of the trial load method which is widely used for the layout of the dam is the program ADSAS. ADSAS allows for temperatures to be applied in two ways. The first represents a uniform change in temperature from the grout temperature. The second is a linear temperature load. This linear load can be used to describe a straight line change in temperature from the upstream to downstream faces. These two methods can be used in combination to apply changes in temperature from the grout temperature as well as temperature

[1] The terms grout temperature and closure temperature are often used interchangeably. They represent the concrete temperature condition at which no temperature stress exists. This is also referred to as the stress-free temperature condition.

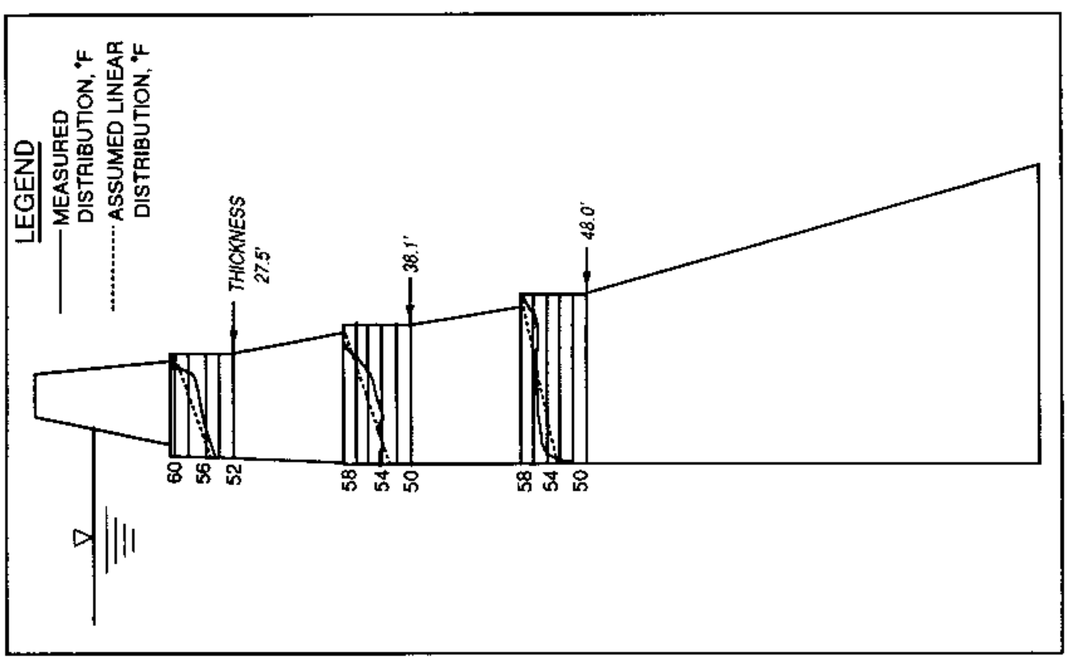


Figure 8-1. Measured temperature distributions in March for a relatively thin dam

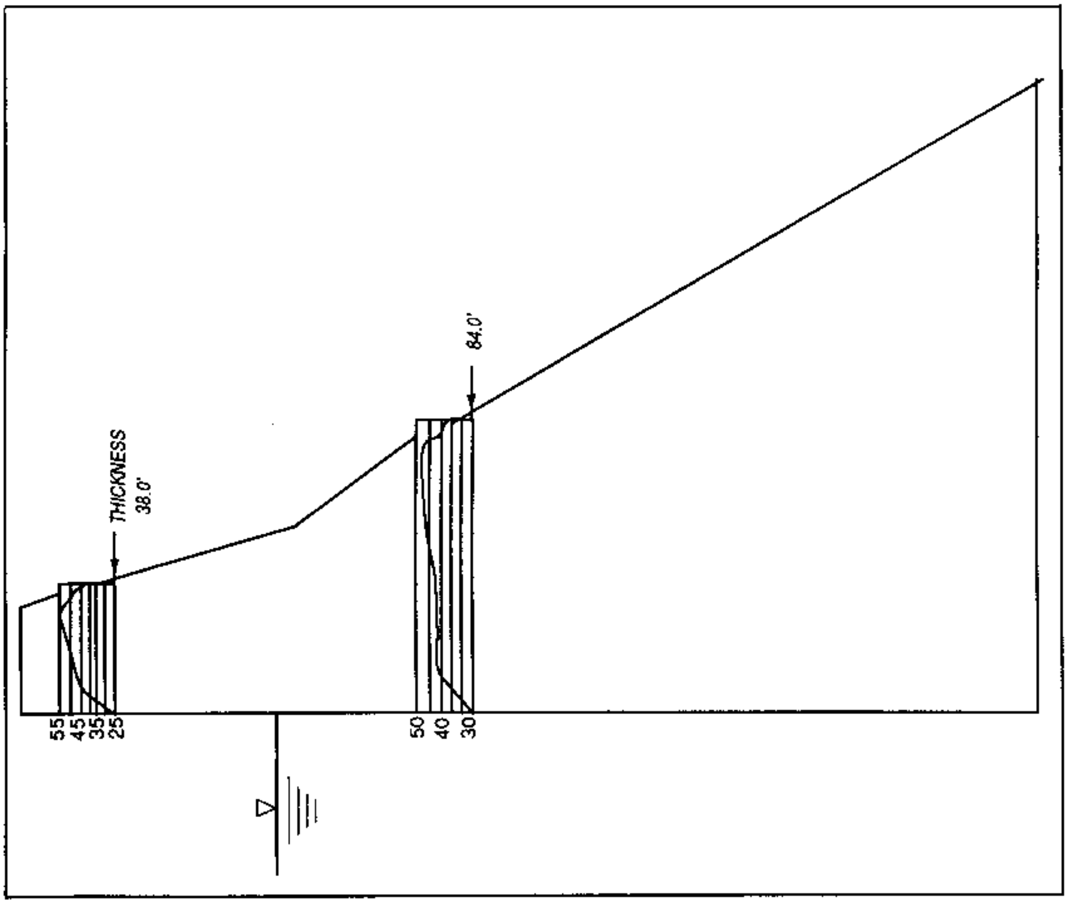


Figure 8-2. Measured temperature distributions in December for a relatively thick dam