Chapter 4
Structural Requirements

4-1. Design Stresses

Allowable stresses will depend on the materials involved, the conditions of loading, and severity of exposure.

a. Allowable stresses. Structural steel and welded joints should be designed in accordance with the allowable stresses outlined in the latest version of the AISC construction manual. Welding details should be as outlined by the latest version of AWS D1.1 "Structural Welding Code-Steel." Gates, Bulkheads, Trashracks, and associated guides should be designed using the allowable stresses outlined in EM 1110-1-2102. Steel and aluminum/switchyard structure should be as designed with the loading and allowable stresses contained in NEMA publication SG-6 "Power Switching Equipment," part 36.

b. Concrete structures. Concrete structures loaded hydraulically should be designed in accordance with the procedures outlined in EM 1110-2-2104 "Strength Design for Reinforced Concrete Hydraulic Structures." Those portions of a powerhouse that will not have water loading, such as the superstructure, may be designed in accordance with the latest version of ACI 318 "Building Code Requirements for Reinforced Concrete."

4-2. Design Loads

a. General considerations. The structures should be designed to sustain the maximum dead, live, hydrostatic, wind, or earthquake loads which may be imposed upon them. Where only partial installation is to be made under the initial construction program, consideration should be given to the temporary loading conditions as well as those anticipated for the completed structures. The stability of all powerhouse monoliths should be investigated for all stages of construction; and loads that may be imposed or absent during the construction period should be accounted for in the design memoranda.

b. Dead loads. Dead loads to be considered in the design consist of the weight of the structure itself, including the walls, floors, partitions, roofs, and all other permanent construction and fixed equipment. The approximate unit weights of materials commonly used in construction can be found in the AISC Manual. For those materials not included, refer to ASCE (1990). A check should be made of the actual weights where a variation might affect the adequacy of the design, or in cases where the construction may vary from normal practice.

c. Live loads. In general, floors are designed for an assumed uniform load per square foot of floor area. However, the floors should be investigated for the effects of any concentrated load, minus the uniform load, over the area occupied. Equipment loads should take into account installation, erection, and maintenance conditions as well as impact and vibration after installation. In most cases, it will be necessary to proceed with the design on the basis of estimated loads and loaded areas until such time as the actual data are available from the manufacturers. All live loads used in design should be recorded with notations as to whether the loads are actual or assumed. The weights of turbines and generators of Corps of Engineers hydroelectric power projects are tabulated in Appendix B1. However, weights will vary considerably for units of the same capacity. Estimates of the weights of the machines should always be requested from the manufacturers for preliminary use in the design. Assumed loads should be checked later against actual loads, and, where differences are appreciable, the necessary modifications in the design should be made.

(1) The live loads shown in Table 4-1 are recommended for the design of slabs, beams, girders, and columns in the area indicated. These loads may be modified, if necessary, to suit the conditions on individual projects, but will ordinarily be considered minimum design loads. These loads may be reduced 20 percent for the design of a girder, truss, column, or footing supporting more than 300 square feet of slab, except that for generator room and erection floors this reduction will be allowed only where the member under consideration supports more than 500 square feet of slab. This differs from ASCE (1990) recommendations for live load reduction because of the loadings historically required in powerhouse floor slabs.

(2) Draft tube decks, gantry decks, and erection bays are often made accessible to trucks and Mobile Cranes, the wheel loads of which may produce stresses greater than those caused by the uniform live load. Under these conditions, the loading used for design should include the weight of the heaviest piece of equipment, such as a complete transformer including oil plus the weight of the truck or crane. Stresses should be computed in accordance with AASHTO "Standard Specifications of Highway Bridges." As a safety measure it would be

Table 4-1
Minimum Uniformly Distributed Live Loads

Element	lb/sq ft
Roofs	50
Stairways	100
Floors:	
Offices	100
Corridors	100
Reception rooms	100
Toilets and locker rooms	100
Equipment and storage rooms	200
Control room	200
Spreading room	150
Generator room	*500
Turbine room	1,000
Erection floor	1,000
Maintenance shop	300
Pump rooms and oil purification rooms	200
Gantry deck (outdoor powerhouse)	300 or H 20
Transformer deck	300
Intake deck--general	300 or H 20
Intake deck--heavy lift areas	1,000
Powerhouse access	300 or H 20
Draft tube deck	300 or H 20

*300/lb/sq ft should be used for mezzanine floors and 1,000 lb/sq ft for areas which may be used for storage or erection of generator or turbine parts

advisable to post the load limit in all cases where such a load is used in design. Where mobile cranes are in use, the design should include outrigger loads. Where the powerhouse monoliths include the headgate structure and intake deck, moving concentrated loads such as mobile cranes and trucks handling equipment parts and transformers should be considered in the design of the deck and supporting structure. In case the deck carries a highway, it should be designed for standard highway loading also.

(3) Impact factors for vehicle wheel loads are given in AASHTO "Standard Specifications for Highway Bridges." Impact factors for crane wheel loads on runways are given in paragraph 4*c*(9).

(4) Wind loading should be applied to the structure as outlined in ASCE (1990). Members subject to stresses produced by a combination of wind and other loads listed under group II in EM 1110-2-2105 should be proportioned on the basis of increased allowable stresses. For concrete structures, ACI 318 and EM 1110-2-2104 provide appropriate load factors to be used for wind loading. The design of switchyard and take-off structures for wind is covered in the reference cited in paragraphs 4-2*d* and 4-15.

(5) Construction loads should be carefully considered to determine if provision should be made in the design of these temporary loads or whether false work or temporary bracing will suffice. It will be noted that construction loads are classified as Group II loading in EM 1110-2-2105, and should have the applicable load factor combinations for concrete structures.

(6) The possibility of seismic activity should be considered and appropriate forces included in the design. The structural analysis for seismic loading consists of two parts: The traditional overturning and sliding stability analysis using an appropriate seismic coefficient, and a dynamic internal stress analysis, using either site dependent earthquake ground motions or a static seismic coefficient. The use of the seismic coefficient should be limited to sites where the peak ground acceleration for the maximum credible earthquake is less than 0.2 g. Where a dynamic analysis is involved the powerhouse should be investigated for both the maximum credible earthquake and the operating basic earthquake. Earthquake motions should be picked by procedures outlined in ETL 1110-2-301 "Interim Procedures for Specifying Earthquake Motions." General guidance for seismic design and analysis is found in ER 1110-2-1806 "Earthquake Design and Analysis for Corps of Engineers Projects." Specific criteria for powerhouses should be as outlined in ETL 1110-2-303 "Earthquake Analysis and Design of Concrete Gravity Dams." This reference should be followed closely when the powerhouse intake forms a part of the dam. Site-dependent earthquake time histories or response spectra should be carefully chosen through geological and seismological investigation of the powerhouse site.

(7) All loads resulting from headwater and from tailwater should be accounted for. Since it is sometimes impracticable to protect the powerhouse against flooding at maximum tailwater elevation, a level should be selected above which flooding and equalization of interior and exterior water loads will occur. This elevation should be determined after careful consideration of all factors involved, particularly the cost of initial construction and of rehabilitation in relation to flood levels and frequencies. The structures should be designed to withstand tailwater pressures up to the chosen level and positive provisions should be incorporated in the structure to permit rapid flooding and equalization of pressures after

the tailwater rises above this level. Provisions should also be made for rapid draining of the powerhouse when the tailwater drops. A maximum tailwater elevation (below that selected for flooding the entire powerhouse) should also be chosen for unwatered draft tubes and provisions for automatic flooding of the water passages when that level is exceeded should be considered. It is nearly always advisable to reduce the uplift pressures on the draft-tube floor by means of a drainage system. When "floating" or relatively flexible floor slabs are used, they are not considered in the stability analysis, either as contributing weight or resisting uplift. When the floor slabs must take part of the foundation load, as is sometimes the case when the foundation is soil or poor rock, uplift should be assumed and the slab made an integral part of the draft-tube structure.

(8) Snow loading should be applied to the structure as outlined in ASCE (1990) "Minimum Design Loads for Buildings and other Structures." Members subject to stresses produced by snow loading should be proportioned by treating it as a Group 1 loading in EM 1110-2-2105 or, in the case of concrete structures, as a basic live loading.

(9) Wheel loads should be treated as moving live loads in the design of crane runways. Maximum wheel loads should be computed from the dead load of the crane and trolleys plus the rated live load capacity, with the load in position to produce maximum truck reaction at the side of the runway under consideration. Dimensional data, weights, and truck reactions for cranes installed in Corps of Engineers powerhouses are given in EM 1110-2-4203. An impact allowance of 10 percent for cranes over 80-ton capacity, and from 12 percent to 18 percent for smaller cranes, should be added to the static loads. Side thrust at the top of the rail should be taken as 10 percent of the summation of the trolley weight and rated capacity, with three-fourths of this amount distributed equally among the wheels at either side of the runway. This may vary in the case of unequal stiffness of the walls supporting the runway. For instance, if one wall is relatively massive, the entire side thrust may be taken by this wall with little or no thrust taken by the more slender wall. The runway design should provide for longitudinal forces at the top of rail equal to 10 percent of the maximum vertical wheel loads. Crane stops should be designed to safely withstand the impact of the crane traveling at full speed with power off. Only the dead weight of the crane will be considered and the resulting longitudinal forces should be provided for in the design of the crane runway.

d. Load on switchyard structures. Switchyard structures should be designed for line pull, equipment load, dead load, wind load, snow load, and ice load in accordance with the requirements of the NEMA Publication SG-6-Power Switching Equipment. Take off tower line loading should conform to ANSI Standard C2, National Electrical Safety Code, Section 25, and shall include applicable combinations of dead load, and line tension due to wind, ice and temperature changes.

4-3. Stability Analysis

a. Outline of investigation. A stability analysis should be made for each monolith of the powerhouse and all critical levels should be investigated for the most severe combination of horizontal and vertical forces. In the case of a monolith in which the power unit will not be installed with the initial construction, the stability should be investigated for the interim as well as the final condition. Analysis should be made for the applicable cases indicated below and for any other combinations of conditions which might prove critical. Cases S-1, S-2, S-3, and S-4, below are applicable when the powerhouse is separated from the dam, and Cases M-1, M-2, M-3, and M-4 are applicable when the powerhouse and headworks form a part of the dam.

(1) Applicable when powerhouse is separated from dam.

(a) Case S-1: head gates open--headwater at top of flood-control pool, hydraulic thrusts, minimum tailwater, spiral case full, draft tube full, uplift, and wind or earthquake.

(b) Case S-2: head gates open, tailwater at powerhouse flooding level, spiral case full, draft tube full, uplift, and wind or earthquake.

(c) Case S-3: head gates closed, tailwater at draft-tube flooding level, spiral case empty, draft tube empty, uplift, and wind or earthquake.

(d) Case S-4 (Construction): no tailwater, and no uplift.

(2) Applicable when powerhouse and headworks, form part of dam.

(a) Case M-1: head gates closed, headwater at top of flood-control pool, minimum tailwater (ice pressure (if applicable)), draft tube and spiral case open to tailwater (uplift), and wind on upstream side or earthquake.

(b) Case M-2: head gates open, headwater at maximum flood level, tailwater at powerhouse flooding level, spiral case full, draft tube full (uplift), and wind on upstream side or earthquake.

(c) Case M-3: head gates closed, headwater at top of flood-control pool, tailwater at draft-tube flooding level, spiral case empty, draft tube empty, uplift, and wind on upstream side or earthquake.

(d) Case M-4 (Construction): no headwater, no tailwater, no uplift, and wind or earthquake.

(3) In some cases, the maximum overturning moment will occur when tailwater is at some intermediate level between minimum and maximum.

(4) In analyzing monoliths containing draft tubes, the floor of the draft tube should not usually be considered as part of the active base area since it is generally designed to take neither uplift nor foundation pressure. (See paragraph 4-2*c*(7)).

(5) Monoliths should also be checked for lateral stability under applicable conditions, and the possibility of flotation at high tailwater levels should be borne in mind.

b. Vertical forces. The vertical forces that should be considered in the stability analysis are the dead weight of the structure, fixed equipment weights, supported weights of earth and water, and uplift. The weights of movable equipment such as cranes and heavily loaded trucks should be included only where such loads will decrease the factor of safety against overturning.

c. Horizontal forces. The horizontal forces that should be considered are those due to headwater, tailwater, ice, earth, and wind or earthquake pressures. Force due to waves should also be included if the fetch is great enough to cause waves of considerable height. The forces resulting from temperature changes in steel penstocks need not be considered, but the pressure of water in the penstocks should be included as hydraulic thrust resulting from wicket gate closure, depending upon the assumed conditions. The application and intensity of wind pressure or earthquake should be as prescribed in paragraphs 4-2*c*(4) and 4-2*c*(6).

d. Uplift assumptions. Effective downstream drainage whether natural or artificial will in general, limit the uplift at the toe of the structure to tailwater. If the powerhouse is separate from the dam, uplift from tailwater should be assumed 100 percent effective on the entire foundation area. If the powerhouse forms part of the dam, uplift assumptions should be the same as those for the dam, as described in EM 1110-2-2200. For those structures founded on soil, uplift should be assumed to vary from headwater to tailwater using the line of seepage method as outlined in EM 1110-2-2502, "Retaining and Floodwalls." For a majority of structures, this method is sufficiently accurate, however there may be special situations where the flow net method is required to evaluate uplift.

e. Base pressures and stability. Ordinarily the maximum base pressures do not govern the design of powerhouse on sound rock. However, regardless of the foundation material, they should always be checked to make sure they do not exceed the safe working values established as a result of the geological or soils investigations. For conditions that include earthquake, the resultant of all forces may fall outside the kern but within the base a sufficient distance so that the allowable foundation pressure is not exceeded. The location of the resultant of all forces, including uplift, acting on the structure should fall within the kern of either a rectangular or irregularly shaped base. In pile foundations, the allowable material, bearing, and tension valves for the piles should not be exceeded. If the foundation at the selected site is entirely soil, or is a combination of soil and rock, special consideration should be given to the possibility of unequal settlement. It may be necessary to investigate the shearing strength of the foundation or, in case of a hillside location, to investigate the stability of the structure and foundation together by means of one of the methods discussed in EM 1110-2-1902.

f. Sliding factor. EM 1110-2-2200 contains the criteria and guidance for assessing the sliding stability for dams and related hydraulic structures. Required factor of safety for major concrete structures are 2.0 for normal static loadings and 1.3 for seismic loading conditions. Horizontal earthquake acceleration can be obtained from seismic zone maps and the seismic coefficient method is the most expedient method to use when calculating sliding stability.

4-4. Subgrade Conditions and Treatments

a. Rock foundation. It is very important that the structure rest on sound material, unweathered and unshattered by blasting, in order to develop full resistance to shearing and sliding. The character of some rock foundations is such that disintegration will take place upon short exposure. In these circumstances it will be necessary to preserve, insofar as possible, the natural characteristics of the unexposed foundation material. Disintegration may be prevented either by delaying the excavation of the last foot or two of material until just prior to placing concrete, or by excavating to final grade and immediately applying asphalt or a similar waterproof coating to the exposed surface. Another method is to place a light concrete cover immediately upon exposure, which provides a better surface for workmen and equipment as well as protection for the foundation. An otherwise sound rock foundation may contain seams of clay or other unsuitable materials which must be excavated and filled with concrete, or areas for broken rock which must be consolidated by pressure grouting.

b. Soil foundation. The design of powerhouse foundation on earth is based on the in situ shear and bearing strength of the underlying soil, with consideration being taken of weak seams at deeper depths below the foundation line. Weak materials may require excavation to firmer material or the use of piles as a foundation. A close cooperation between the designer and the foundation and materials engineer must exist even in preliminary design. Factors of safety against sliding should be computed using procedures discussed in paragraph 4-3*f*.

4-5. Foundation Drainage and Grouting

a. Rock foundation. Provisions should be made for foundation drainage, particularly under the draft-tube floor slabs, to reduce uplift and permit the unwatering of draft tubes. Usually, a network of drains under the draft tubes is all that is required. Holes drilled into the rock connect with these drains, which discharge through weep holes in the slab into the draft tubes. Drain holes should be cleared on a routine basis, perhaps every 5 years, to ensure their functional capability. For unwatering, a drain in each draft tube leads, through a valved connecting pipe, to a header which drains to a sump from which the water is pumped outside the powerhouse. Theoretically, the drill holes in the rock should be deep enough so the hydrostatic uplift (due to maximum tailwater with draft tube unwatered) on a horizontal plane at the bottom of the holes will be more than balanced by the weight of the rock above the plane plus the weight of the draft tube floor slab. A lesser depth will usually be satisfactory, as the rock may be assumed to “arch” to some extent across the end piers of the draft tube. It is recommended that the drain holes extend to a depth at least equal to one-half the monolith width below the floor slab. Drain holes should be spaced about 12 feet to 15 feet on centers with weep holes in the slab 6 feet to 7-1/2 feet on centers. Where the nature of the rock indicates percolation of such magnitude as to render the unwatering of draft tubes difficult, perimeter grouting, area grouting, or both may be used within the powerhouse foundation area. Care should be taken that this grouting does not interfere with drainage essential to the dam or the powerhouse. If perimeter grouting is used, a system of relief drains near the upstream side of the monoliths is necessary to prevent possible building up of headwater pressure under the structure in case of leakage through or under the upstream grout curtain. Because of the possibility that headwater may enter the area and cause worse unwatering and uplift conditions than would have been the case without curtains, it is desirable to avoid perimeter grouting if possible. For additional information on the purpose, theory, and methods of foundation grouting refer to EM 1110-2-3506.

b. Soil foundations. When powerhouse structures rest on soil, it is necessary to protect the foundation material against scour and piping. The potential for effective drainage and grouting in soil materials is very sensitive to the exact nature of the material. Uplift reduction may be more effective if underlying drainage blankets are used rather than drain holes.

4-6. Substructure Functions and Components

The powerhouse substructure supports the turbines and generators as well as a superstructure for their protection and equipment related to their operation. The substructure also contains the water passages, includes rooms and galleries needed for certain mechanical and electrical equipment and services, and furnishes most of the mass needed for stability. It is usually desirable, where the turbines have steel spiral cases, to provide recesses in the substructure for accommodation of these parts and to design the structure so that concreting operations can continue without interruption during their installation. Access galleries to the draft tube and spiral case should also be provided in the substructure. Except as provided in Table 4-2, substructure concrete should be placed in lifts, generally not more than 5 feet thick. Each lift should be divided into pours by vertical joints as determined by equipment installation needs and as required to minimize shrinkage and temperature cracking.

Table 4-2
Powerhouse Concrete Lift Height Limitations (in feet)

Type of Placement	Temperature-Controlled Concrete Watertight	Temperature-Controlled Concrete Other	Normal Concrete Watertight	Normal Concrete Other
Mass or semi-mass areas such as draft tube or spiral case roofs	7-1/2	7-1/2	5	5
Walls and piers over 7 ft thick	7-1/2	10	5	10
Walls and piers 5 ft to 7 ft thick	7-1/2	15	5	10
Walls 18 in. to 5 ft thick	15	20	10	20
Walls less than 18 in. thick heavily reinforced		10		10
Walls less than 18 in. thick moderately reinforced		15		15

4-7. Joints

a. General. The purpose of joints is to facilitate construction, to prevent destructive or unsightly cracks, and to reduce or eliminate the transmission of stresses from one portion of a structure to another.

b. Types of joints. Joints may be classified as expansion, contraction, construction, or control. Selection of the location and type of joint is governed by both architectural and structural requirements. Reinforcing steel or structural steel should not cross expansion or contraction joints, but may be continued across construction and control joints. The functions of the various joints are as follows:

(1) Contraction joints are used to divide the structure into separate monoliths, the principal purpose being to reduce the tendency to crack due to shrinkage resulting from the cooling of the concrete from the maximum temperature. The location and spacing of the transverse contraction joints will be determined by the space required for the unit. Where the powerhouse structure is located immediately downstream or adjacent to the concrete gravity dam, a contraction joint will be provided to separate the dam and powerhouse. In the above case where more than one generating unit is involved, the spacing of the transverse contraction joints of the intake monoliths must be the same as in the powerhouse substructure, although not necessarily on the same alignment. Other detailed criteria for contraction joints as well as the necessary construction joints are given in EM 1110-2-2000. Ordinarily, no initial opening or treatment of the vertical concrete surfaces at the joint is necessary. However, the longitudinal formed joint between the toe of the dam and the powerhouse (see paragraph 6) should have an initial opening of about 1 inch filled with a suitable premolded compressive-type filler to permit possible movement of the dam without transfer of load to the powerhouse substructure. Contraction joints in the substructure should continue through the superstructure. Offsets in contraction joints are undesirable and should be avoided if possible.

(2) Construction joints are required primarily for the practical purpose of dividing the structure into satisfactory and convenient working units during concrete placement. Also, in large or irregular pours, it is usually desirable to require construction joints in order to minimize the influence of shrinkage on the formation of cracks. Construction joints should be so located and designed that they will not affect the continuity of the structure. Reinforcing steel should be continued across the joint and provisions made to transmit any shear from one side to the other. Horizontal construction joints normally do not require keying, because the roughened surface resulting from water jetting, greencutting, or sand blasting is adequate for transferring shear. This type of joint preparation is not feasible for vertical construction joints where shear keys are usually necessary.

(3) Control joints are adequately described and detailed in EM 1110-2-2000 and in Guide Spec. CW-03301.

c. Criteria for location of construction joints. Construction joints should be located to minimize cracking in the more massive concrete placements. Greater restrictions are necessary where concrete is to be watertight, such as in draft tubes and spiral cases; cracking is caused by heat generated during curing of the concrete and external and internal restraint to attendant volume changes. This type of cracking is minimized by reducing lift heights, using low slump mixes, replacing cement with pozzolan, increasing cure time between lifts, insulating to control the rate of cooling, and reducing the placing temperature of the concrete. Concrete placed under these conditions is termed "temperature-controlled concrete."

(1) For less massive concrete placements, construction joints should be located to facilitate forming and placing of concrete. Lift height is controlled by form, shoring, and bracing design requirements to resist the concrete pressure and dead weight. Reduced lift heights are often necessary when concrete placement is made difficult because of heavy, closely spaced reinforcing, or other physical constraints to placing and compaction equipment.

(2) Basic lift heights (in feet) should not exceed the limitations shown in Table 4-2.

(3) Powerhouse substructures often use a two stage concreting operation where the downstream wall and tailrace structure, and some or all of the spiral case piers are placed in the first stage concrete forming a skeleton structure. The embedded turbine parts, and the sloping floor and roof of unlined concrete spiral cases are cast in the second stage concrete. This arrangement allows early "water-up" of the project, and also allows each unit to be placed "on-line" upon its completion while construction continues in neighboring bays. Keys should be formed in the vertical construction joints of the first stage concrete, and reinforcing dowels provided so the completed structure acts monolithically. Where the vertical joints are subjected to headwater or tailwater pressure, reinforcing should also be adequate to resist the hydrostatic loading created in the joint. When embedding steel lined spiral cases, the lift heights below the center line of the distributor lift heights are limited to 5 feet.

(4) To prevent distortion of the turbine liner concrete should be placed in layers such that there is no more than a one foot height differential of fresh concrete against the liner. The liner is to be continuously sprayed with water during cure of the concrete.

(5) Where main structural slabs frame into walls, it is preferable to locate a horizontal construction joint in the wall at the elevation of the bottom of the slab. The slab is then cast over the prepared wall joint. Keying of mainslabs into walls or piers pose design and construction problems, and should be used only when the construction schedule dictates a need for delaying the slab placement. When main slabs are keyed, it is necessary to dowel heavy reinforcing through the forms and then lap splice closely spaced slab reinforcing at a point of maximum stress. Deep key ways interfere with the vertical curtain of wall reinforcing particularly if it is necessary to waterstop the keyed joint. These problems are less evident in thin, lightly loaded slabs, and it is often more economical to key these slabs into the walls.

(6) Exterior concrete decks covering interior areas required to be dry should have a minimum thickness of 12 inches. Minimum reinforcement should be 0.75% of the cross sectional area with half distributed to each face. Waterstops should be provided at contraction joints, and construction joints should be treated as specified in guide specifications CW 03301.

(7) Vertical construction joints should be used to divide lift placements covering large areas into two or more smaller placements based on the following:

(a) Maximum rate concrete can be batched and placed without developing cold joints.

(b) Watertight concrete or other serviceability requirements affected by shrinkage cracking.

(c) Openings, blockouts or other discontinuities in the placement that tend to generate cracks.

(d) A need for a vertical construction joint, such as the one normally used between the intake and powerhouse structures, to allow flexibility in concrete placing schedules for different construction areas.

(e) The temperature-controlled concrete requirements.

(8) As a general guide, base slabs up to 100 feet wide, can be placed, without need for intermediate vertical joints, using a 3-inch aggregate, temperature-controlled concrete, and a batching and placing capability of 150 cubic yards per hour. Because of the tendency for shrinkage cracks to radiate out from the turbine pit blockout, the greatest dimension for a spiral case roof

pour should be limited to 70 feet, using 3-inch aggregate temperature-controlled concrete. By using additional reinforcing to minimize crack widths for satisfying the watertight concrete requirements, pour widths can be increased. With the added reinforcing, and by carefully establishing the temperature control requirements, it is seldom necessary to resort to segmented, waterstopped roof placements used in the past for unlimited concrete spiral cases.

(9) When vertical construction joints are required in the substructure, they should extend upward through the massive part of the structure, but need not extend into less massive piers, slabs or walls. Vertical construction joints should be keyed, and adequate shear friction reinforcing should be provided across the joint to develop the required shear capacity.

(10) A sloping construction joint should be located at the top of the main intermediate piers in the intake and draft tube. The roof of the water passage is then placed across the top of the prepared surface of the piers. Due to the slope of the roof, several horizontally placed lifts are usually required to complete the roof. Where the lift tends to feather out to the roof form, the pour line should be dubbed down 12 inches to eliminate the feathered edge.

(11) Consideration should be given to the location of horizontal construction joints on exposed faces. A V-notch rustication can be chamfering the joints in keeping with the architectural treatment.

4-8. Waterstops

Waterstops across contraction joints are necessary to prevent leakage and obtain satisfactory dry operating and working conditions. They are required to exclude water under head in the substructure and to ensure weather-tightness of the joints in the superstructure. Material of rubber or polyvinylchloride (PVC) is suitable for this purpose. Extensive experience in the use of molded rubber or extruded polyvinylchloride waterstops in joints of conduits and hydraulic structures have proved the practicability and advantages of using either of these materials. Copper waterstops were used in the past, however, they will fail where yielding foundations or other conditions result in differential movement between monoliths. They are also easily damaged during installation. PVC or rubber waterstops with a center bulb can withstand this type of movement and are recommended for use in hydroelectric products. A wider width is indicated for waterstops in the substructure where large aggregate is used and higher water pressures exist than for waterstops to be installed in low-pressure areas or for weather-tightness only. Waterstops should be placed as near to the surface as practicable without forming weak corners in the concrete that may spall as a result of weathering, or impact, and should create a continuous barrier about the protected area. All laps or joints in rubber waterstops should be vulcanized or satisfactorily cemented together, and joints in “PVC” waterstops should be adequately heat sealed. Waterstops in contact with headwater for structures founded on rock should terminate in a recess formed by drilling holes 6 inches deep into the rocks and should be carefully grouted in place. Occasionally, double waterstops are required in pier joints, one on either side of a formed hole, containing bituminous material. In some important locations, two waterstops and a drain should be used to ensure water-tightness.

4-9. Draft Tubes

The outline of the draft tube is usually determined by the turbine manufacturer to suit the turbine operating requirements. However, in most cases, the manufacturer will be limited by certain physical requirements, such as the spacing and setting of the units, depth of foundations, and elevation of tailrace. The draft-tube portion of the substructure should be designed to withstand all loads that may be imposed on it, including superstructure loads, foundation reactions on the piers, tailwater on the roof, and the bursting effect of tailwater inside the draft-tube. Uplift under the floor of the draft-tube should also be considered in the design of the slab even when relief drains are provided. The upstream ends of intermediate piers should have heavy cast or structural steel nosing (usually furnished by the turbine manufacturer) to withstand the concentrated vertical load and to protect the piers from erosion. Piers between adjacent draft-tubes are usually bisected by the monolith contraction joint from which water is excluded by seals near the gate slot. Therefore, each half pier must be designed for the pressure of tailwater on the inside of the draft tube as a normal condition. It is advisable to consider also the possibility of unbalanced load in the opposite direction in case of failure of a contraction joint seal with the draft-tube unwatered.

4-10. Spiral Cases

a. General. Spiral cases should be designed to withstand the bursting pressure of maximum headwater plus water hammer.

b. Types of spiral cases. The type of spiral case depends on the power plant being considered.

(1) For low-head plants they may be of unlined concrete with engineered reinforcement to withstand applied dead, hydraulic and equipment loads.

(2) For medium and high head plants, they should be made of steel plate with shop welded longitudinal joints. Circumferential joints may be either field welded or high strength bolted, depending on the turbine manufacturers design. Welded joints should be double-vee butt joints made under strict quality control and in accordance with the provisions of ASME. It is preferred that the "c" sections of spiral cases requiring field welding be butt-welded to skirt plates which should be shop-welded to the stay rings. All longitudinal welds should be radiographed. Ordinarily, stress relieving will not be required. When considering spiral cases under high head, and when shipping, handling, and erection cost would control, consideration should be given to the use of high strength steels. Completed spiral cases should be proof tested hydrostatically with a test pressure equal to 1-1/2 times the maximum design pressure.

c. Construction details.

(1) Consideration should be given to under-drainage of the turbine floor to intercept seepage upward through the spiral case roof. Under-drainage should consist of a grid of shallow trenches in the concrete subfloor covered by porous concrete planks, and overlaid by a 3 inch gravel bed. The vertical joint between the intake structure and the spiral case roof should contain a double waterstop and drain. Where the spiral case piers are placed in the first of a two-stage concreting operation described in paragraph 4-7*c*, and extend above the spiral case roof line, the joint between the piers and roof should be double waterstopped. Consideration should also be given to providing a grouted contact strip in the contraction joint between adjacent spiral case piers at approximately mid-height of the pier. When a unit is unwatered, unbalanced hydrostatic load will be shared by both piers. The contact strip is constructed by injecting grout into a formed recess about 3 inches wide and 12 inches high located in the contraction joint. A waterstop should be located just above and below the recess to prevent grouting the entire joint.

(2) The substructure may be "skeletonized" and the downstream wall and portions of the side walls or piers completed prior to embedment of the spiral case. In this case a minimum clearance of 2 to 3 feet must be left between the spiral case liner and the concrete walls of the recess. The transition section from penstock to spiral case extension should not be encased. A penstock room or gallery should be provided to house the transition, penstock coupling, and when required, a butterfly or spherical closure valve. A Dresser-type coupling should be used to connect the penstock to the spiral case. The lifts of concrete around the spiral case liner should be limited to the depth specified in paragraph 4-7*c*. A minimum of 72 hours per lift shall elapse between the placing of each successive lift. Bent steel 'J'-pipes 6 inches or more in diameter should be provided for placing concrete under the stay ring, discharge ring, and spiral case. The number of 'J' pipes required depends on the size of the spiral case. Concrete may be pumped through the 'J' pipes using a positive displacement concrete pump. After concrete placement is complete the 'J' pipes should be filled with concrete and left in place.

d. Embedment conditions. Two methods of embedment of steel spiral cases in concrete are commonly used:

(1) When steel spiral cases are to be embedded in concrete in an unwatered condition, the top portion of the spiral case should be covered with a compressible membrane to ensure that the spiral case liner resists internal pressure by ring tension with only a small load being transmitted into the surrounding concrete. The compressible membrane should consist of sheets of closed cell foam material with the property that a 1/4-inch-thick piece deflects 0.10 inch under a 50-psi uniform pressure applied normal to the surface. Polyvinylchloride foam and polyurethane foam are acceptable, and the sheets of this material should be attached to the spiral case liner with an adhesive. The thickness of the membrane depends on the diameter and thickness of the spiral case liner, and the internal pressure being resisted. The compressible membrane should extend to the first construction joint below the horizontal centerline of the spiral case. A drain should be provided along the lower limits of the compressible membrane in order to prevent transmitting stress to the concrete through a water medium.

(2) When steel spiral cases are to be embedded in concrete under a pressurized condition, a test barrel is used to close off the opening between the upper and lower stay rings, and a test head is attached at the inlet of the spiral case extension. The spiral case is filled with water and pressurized to the test pressure or to a pressure equal to head water plus water hammer while encasement concrete is placed. Grout and vent holes in the stay ring are fitted with plugs which remain in place until the spiral case is unwatered. After concrete has been placed

at least one lift above the top of the spiral case, and the top lift has set at least 72 hours, the spiral case should be unwatered and the test barrel and test head removed. Workable concrete should then be placed through the bent steel 'J' pipes to fill the void under the stay ring/discharge ring. Neither a compressible membrane or drains are required for embedding spiral cases under the pressurized condition, except when low alloy steels or high strength quenched and tempered alloy steels are used for spiral case construction, consideration should be given to providing drains.

e. Concrete placing. The use of mechanical vibrators will not be permitted closer than 5 feet to any part of the spiral case, stay ring/discharge ring, or draft tube liner, except that small vibrators may be lowered through the grout holes in the stay ring to vibrate concrete placed through the steel 'J' pipes.

(1) Concrete placement through the steel 'J' pipes should be accomplished in a single lift operation which brings the concrete to within about 1 inch of the lower stay ring. After a minimum seven day cure period, grout fill pipes shall be attached to grout holes spaced evenly around the stay ring, and non-shrink grout injected at a head not exceeding 6 feet. A vent hole shall be located approximately midway between grout holes to permit escape of entrapped air and water.

(2) Concrete reinforcement above the spiral case should be designed to distribute the generator pedestal and turbine floor loads to the stay ring and the spiral case piers and walls.

4-11. Generator Pedestals

The generator, except in some low-head plants, is usually supported on a heavy concrete pedestal. The details of this pedestal will depend on the make and type of generator to be installed. It should be of massive construction and should be designed to resist vibrational forces from the moving mechanical parts, the heavy loads from the thrust bearing, and the short circuit torque of the generator. It is usually designed to support the generator room floor also. Openings in the pedestal should be provided on all four sides when practicable for access to and ventilation of the turbine pit. Adequate head room should be provided between the underside of the generator and the generator platform, if one is used, and between this platform and the turbine walkway.

4-12. Bulb Turbine Supports

Bulb turbines are supported in a much different manner than the typical vertical shaft Francis or Kaplan unit. A typical cross section of a bulb unit is shown in Figure 4-1. The main element of support for a bulb turbine is the stay column. It must carry the weight of the rotating parts, most of the stationary parts, and the hydraulic loads due to thrust, hydrostatic pressure, and transient loading. A typical stay column consists of an upper and lower support column fixed to the inner and outer stay cones. The inner stay cone forms the inner water passage and houses the turbine parts. The outer cone forms the outside water passage surface. This cone and both support columns are embedded in concrete, thereby transmitting all loads to the structure.

4-13. Types of Superstructures

The main superstructure may be one of three types: indoor, semi-outdoor, or outdoor. The indoor type completely encloses the generators and the erection bay and has an inside crane runway supported by the walls of the structure. The semi-outdoor type consists of a continuously reinforced slab over generators and erection bay supported by heavy transverse walls enclosing 2 or 3 units. The powerhouse crane is an outdoor gantry with one runway rail on each side of the low superstructure. Sliding hatches in the roof over each generator and in the erection bay provide access for handling equipment with the crane. In the outdoor type, each generator is protected by a light steel housing which is removed by the outdoor gantry crane when access to the machine is necessary for other than routine maintenance. The erection and repair space is in the substructure and has a roof hatch for equipment access. Choice of type should be dictated by consideration of first cost of the structure with all equipment in place, cost of maintenance of building and equipment, and protection from the elements. The indoor type affords greatest protection from the weather and facilitates operation and maintenance of equipment. The semi-outdoor type may sometimes have a marginal economic advantage insofar as it pertains to the cost of the structure, but this advantage will not always offset the increased equipment cost. The outdoor type is structurally the most economical at sites with low maximum tailwater. The structural economy must, however, be weighed against increased cost of generator housing, greater crane costs, and increased maintenance of equipment.

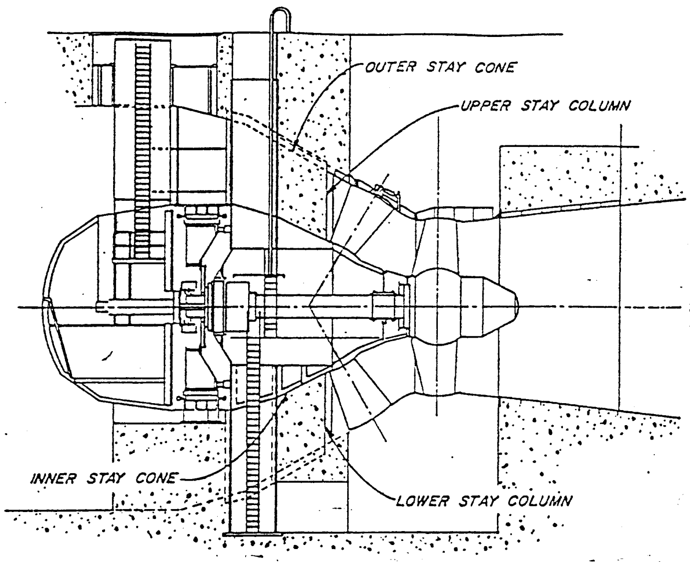

FIG. 4-1. TYPICAL CROSS SECTION THRU BULB UNIT.

Figure 4-1. Typical cross section thru bulb unit

4-14. Superstructure - Indoor Powerhouse

a. Framing. The framing of the superstructure may be cast-in-place or precast reinforced concrete, prestressed concrete, structural steel, or a combination. The choice is dependent on economy and architectural requirements.

b. Concrete walls. Concrete exterior walls may be cast-in-place with uniform thickness, column and spandrel wall, precast panels, or prestressed precast units. They must all be designed to withstand the stresses from possible loading combinations and at the same time provide the necessary space requirements, both for embedded items and interior clearances. Provisions must usually be made for carrying loads imposed by bridge cranes and the rails are usually supported by continuous corbels or wall offsets at the elevation of the rails. Exterior concrete should not be rubbed nor should form liner be used, but a rigid specification for forming should be set up to insure reasonably smooth, plane surfaces. In some cases, special finishes, such as bush hammered or striated concrete, may be considered for special architectural effect. Matched tongue-and-groove lumber of 2-inch nominal thickness is satisfactory for sheathing. Wall pours should not be made more than about 10 feet in height. Horizontal rustications should be used at the pour joints on the exterior side where practicable in order to prevent unsightly spalling; hence, locations of joints and pour heights will depend partly on the exterior architectural treatment. In the interior of the powerhouse where neat appearance is essential, it will be necessary to provide a smooth, dustless surface. The requirements for obtaining such a surface are contained in EM 1110-2-2000. Sack rubbing of the concrete surfaces should be avoided insofar as practicable. Contraction joints in the same vertical plane as those in the substructure should be provided in the superstructure walls. Criteria for openings in exterior walls are given in paragraph 3-1.

c. Steel framing and walls. While self-supporting concrete walls are usually preferred in the generator and erection bays, their justification is dependent upon economy and tailwater limitations. Where economically feasible, steel framing with curtain walls or insulated panels should be used where the generator floor is above the maximum tailwater. The use of steel framing may, in some cases, be desirable to permit the early installation and use of the crane runways and crane. This advantage should, however, be evaluated only in terms of overall cost. The steel framing for each monolith should be a separate unit, with no steel except the crane rails crossing the contraction joints. Crane rail splices should be staggered a few inches with the joints. One bay in each monolith should be diagonally braced in both roof and walls. Bents may be composed of columns supporting simple beams or girders or may be rigid frames. Trusses should be used only if dictated by unusually heavy loads and long spans. In case it will usually be advantageous to weld shop connections and bolt field connections. Exceptions should be made for field splices in long rigid frames, which should be welded for structural continuity.

(1) If steel framing is used in other parts of the structure, it is usually of the conventional beam and column type used in office buildings.

(2) Suspended ceilings should be avoided unless economically justifiable. Inner tile walls to conceal the steel columns or rigid frames should be avoided. Not only is concealment of the steel considered unjustified but high thin walls are a hazard structurally.

d. Floors. Floors systems should be reinforced concrete flat slabs or one or two-way slabs with a 6-inch minimum thickness. The types of floor finishes and cove details designated for the various parts of the superstructure are given in paragraph 3-4*a*. Placement of concrete in floor slabs should be stopped sufficiently below finish grade to allow for appropriate finishes. The thickness of the structural slab will in many cases be determined by the member and size of electrical conduits which it must encase. Separate concrete fill placed on top of the structural slabs to encase conduits is uneconomical but can be used where large numbers of conduits must be accommodated, or where reinforcement in the structural slab is closely spaced. If used, it is recommended that the generator room floor be designed as a slab of uniform thickness, supported on the upstream and downstream walls and at the generator pedestal and carried on double columns at the contraction joints. Building contraction joints with water stops should be carried through all floor slabs. Shrinkage and temperature steel should be provided in the tops of all slabs. At hatchways through floors, flush sockets should be provided for the installation of temporary railings for protection of personnel at times when the covers are removed.

e. Roofs. Roof framing will usually consist of precast, prestressed units such as tee's, double tee's or hollow core plank. If structural steel framing is used, it will usually be fabricated steel girders supporting steel purlins, which in turn, support the roof deck. Slope for drainage should be provided by the slope of the roof deck or the use of sloped insulation. The use of lightweight or sawdust concrete should be avoided. Insulation, embedded in hot bitumen or mechanically fastened should be applied over a vapor seal course to the roof slab or deck. Foam insulation should be avoided due to the unevenness of application. Thickness of insulation should be determined by an analysis of heating and cooling requirements. Roofing criteria are given in paragraph 3-2.

f. Future extensions. A temporary end wall must be provided for a superstructure which will at some future time, be extended to house additional units. The construction of the temporary wall should be such that it may be easily removed, and with a minimum of interference in the operation of the station. The temporary wall below the maximum tailwater elevation should be made of precast concrete slabs, supported on a steel framework, designed to resist the tailwater pressure, and sealed with rubber or polyvinylchloride water seals at all joints. The remainder of the temporary wall could be made of prefabricated metal panels which can be removed and possibly utilized in the future permanent end wall.

g. Vibration. The superstructure in the generator monoliths will be subject to vibration caused by the generating units. In order to minimize the effect of vibration on the main structural framing, the superstructure should be made as rigid as practicable. Concrete columns and walls should be integral with floor slabs and girders. Steel beams should be framed into columns and girders with full depth connections and not with seat angles alone. In some cases it may be desirable to use top flange clip angles also. Special attention should be paid to framing connections in light floors, balconies, stairways, and roofs and to fastenings of gratings, prefabricated metal panels, precast slabs and handrails. Threaded or welded handrail connections are preferred to pin connections. The effect of vibration in a generator monolith is, of course, greater on members close to the unit than on those at a distance. Also, members in parts of the structure separated from the generator monoliths by contraction joints will be less affected. Therefore, the designer must use careful judgment in determining the extent to which vibration will influence the design of such members.

4-15. Intakes

a. Type of intakes. Intakes may be classified as low pressure, or high pressure, according to the head on the inlet, but there is no definite line of demarcation separating the two types. For low-head plants and for developments where the pool drawdown is small, low-pressure intakes are used. If the pool drawdown is to be large, such as on many multipurpose projects, the intake will be of the high-pressure type. It is advantageous to locate the intake high as practicable in order to minimize weights and travel distance of gates, size of hoist, etc., as well to keep the sill above possible silt deposits. Low-pressure intakes are usually incorporated in the dam and, for low-head plants, are also part of the powerhouse structure. High-pressure intakes may be in the dam itself or may be in a separate structure or structures in the forebay. The essential requirement if the two types are the same, but the details and equipment may be radically different. Features common to practically all intakes are: trash racks, gates, steel bulkheads, concrete stoplogs, or all three, and converging water passage or passages.

b. Shape of intake. The lines of the intake should be carefully laid out to obtain water velocities increasing gradually from the racks to the penstock, or to the spiral entrance. Abrupt changes in area of the water passage should be avoided in order to minimize turbulence and consequent power loss. The sections between the rectangular gate and the round penstock entrance is particularly important. The transition is ordinarily made in a distance about equal to the diameter of the penstock. Model tests are of great value in determining a satisfactory shape of intake, especially if Juvenile Fish Bypass is a design consideration.

c. Trash racks. Trash racks are usually vertical in order to economize on length if intake structure. For low-head intakes, however, where the increase in length of structure would be small or where considerable trash accumulation may be expected, they are often sloped to facilitate raking. Water velocities at the racks should be kept as low as economically practicable with a maximum, for low-pressure intakes, of about 4 feet per second. For high-pressure intakes, greater velocities are permissible but should not exceed about 10 feet per second.

(1) The racks are usually designed for an unbalanced head of 10 to 20 feet of water and are fabricated by welding in sections of a size convenient for handling. For low-head intakes, stresses due to complete stoppage and full head should be investigated and should not exceed 150 percent of normal stresses. If the racks are to be sheathed for the purpose of unwatering the intake, case II working stresses should not be exceeded for that loading condition. The clear distance between rack bars varies from two to six inches or more, depending on the size and type of turbine and the minimum operating clearances. Bar thickness should be consistent with structural design requirements, with the vibrational effects resulting from flowing water being considered. A thick bar should be used with the depth of the bar controlled by the allowable working stress.

(2) The design of the guides and centering devices for the rack sections should receive careful attention. Clearances should be small enough to prevent offsets interfering with removal of the racks, or operation of a rake if one is provided. Embedded members on the guide slots should have corrosion-resisting exposed surfaces. Corrosion-resisting clad steel is satisfactory for the purpose.

(3) For high-pressure intakes in concrete dams, the trash rack supporting structure is sometimes built out from the face of the dam in the form of a semicircle in order to gain rack area to maintain low velocities.

d. Gates. Provisions for emergency closure of the intake downstream from the racks is necessary to protect the generator unit. A vertical lift gate in each water passage is usually provided and is normally suspended just above the roof of the intake from a fixed hoist. On very low-head multiunit plants a single set of intake gates operated by a gantry crane is adequate and will be less expensive than individual gates operated by fixed hoists. In either type of installation, self-closing tractor gates capable of operating under full flow are provided. Fixed wheel gates may be the most economical type for intakes where a gantry crane will be used for operation. Bronze-brushed wheel bearings should be used if the wheels will be submerged when the gate is in the stored position, otherwise, antifriction bearings can be used. For all but the lowest head intakes, “caterpillar” type gates with corrosion-resisting steel rollers and tracks have been found to be the most economical.

(1) In selecting the position of the gate slots, limiting velocities of flow as well as economical gate size should be considered. If the slots are located too far downstream, where the opening is small, power reduction from eddy losses may be more costly than a larger gate. The duration of peak demand on the plant will also affect the location of the slots, since a higher velocity may be tolerated for a short time. In any case, however, it is advisable to keep the maximum velocity V in feet per second below that given by the expression:

$$V = 0.12\sqrt{2gh}$$

in which h is the head on the center line of the gate at normal power pool.

(2) Slots for stop logs or bulkhead gates are usually provided just upstream from the gates so the gate slots may be unwatered for maintenance operations. In case where headwater is never far above the top of the intake, the racks are sometimes designed to support sheathing for unwatering.

(3) Essential fixed metal in the slop-log slots should have corrosion-resisting exposed surfaces, since these slots cannot be easily unwatered for repairs.

e. Air vents. Since emergency closure must be made under full flow conditions, negative pressures will tend to buildup at the top of the intake just downstream from a downstream-sealing gate as the gate is lowered. To prevent excessive negative pressures from occurring in the penstock during emergency gate closure and to exhaust air during penstock filling operations, one or more air vents should be provided just downstream from the gate. The air vents should be of sufficient size to maintain a pressure of not less than 1/2 atmosphere in the penstock at the maximum rate of depletion of water from the penstock under emergency closure conditions. The opening in the intake roof should be as close as practicable to the gate. The upper end of the air passage should be open to the atmosphere well above maximum headwater and in a location not readily accessible to personnel. Gates sealing on the upstream side are sometimes used. Air vents in the penstock may then be eliminated, as enough air will be introduced into the water passage through the opening between the downstream side of the gate and the concrete structure. Gate slots for both the upstream and downstream seal gates should be adequately vented by the use of open grating covers or by other means.

f. Prevention of ice troubles. Periods of freezing weather are likely to cause trouble with ice at low-pressure intakes, and the design of plants in northern latitudes should take this into account.

(1) Frazil and anchor ice may cause loss of head by forming on, or clogging rack openings, or may immobilize racks and gates by massing in the slots. Continuous surface ice tends to prevent the formation of frazil and anchor ice, and for this reason an ice sheet in the forebay is more beneficial than otherwise, except as it interferes with raking, or at breakup when it must be chuted to the tailrace or passed over the dam.

(2) Formation of ice may be prevented in the slots by means of electric heaters in casings embedded in the piers next to the guides and on the racks by a bubbler system with outlet nozzles just below the bottom of the racks and far enough upstream for the released air to carry the ice particles to the surface without coming in contact with the racks. At the surface the ice is sluiced to the tailrace.

4-16. Penstocks and Surge Tanks

a. Details and design. The determination of the diameter of penstock and the selection of size, type, and location of surge tank, if one is used, involve rather complex economic considerations. Therefore, only details of design will be discussed.

b. Free standing penstocks and surge tanks. The penstock should be designed for full pressure due to static head caused by maximum elevation of the operating range for the intake pool plus waterhammer. Waterhammer studies should be conducted to determine transient pressures at any point along its length. The following design conditions and their corresponding allowable stresses for carbon steels (see ASME, Boiler and Vessel Pressure Code, Sections 8 and 9 when other steels are used) are to be considered for these structures.

(1) This condition includes maximum, minimum, and rated turbine static heads plus waterhammer due to normal operation, load rejection and load acceptance. It also includes stresses due to gravity loads and longitudinal stresses due to penstock movement. The allowable stress for this basic condition is equal to the smaller of 1/4 of the specified tensile strength or 1/2 of the specified yield strength. The load acceptance condition includes minimum static head and loading the turbine from speed no load to full gate opening at the maximum rate of gate opening. This condition will indicate a minimum pressure grade line for the determination of sub atmospheric conditions. Amstutz buckling criteria should be used for embedded conduits. Stewart buckling criteria should be used for non-embedded conduits. If a valve is used as an emergency closure device, the conditions at maximum flow and maximum head with maximum valve closure rate must be analyzed.

(2) This includes conditions during filling and drainage of the penstock or surge tank and seismic loads during normal operation. The allowable stress for this condition is equal to 1/3 of the specified tensile stress or 2/3 of the specified yield point, whichever is less.

(3) This condition includes the governor cushioning stroke being inoperative and partial gate closure in 2 *L/a* seconds at a maximum rate, where *L* equals the conduit length in feet and *a* equals the pressure wave velocity in feet per second. The allowable stress for this condition is equal to 1/2 of the specified tensile strength, but in no case shall this stress exceed the specified minimum yield stress.

(a) After combining longitudinal and circumferential stresses in accordance with the Hencky-Mises Theory, where $S_e^2 = S_x^2 + S_x S_y + S_y^2$, the allowable stresses are not to be exceeded by the resulting equivalent stress at any point on the penstock or surge tank.

(b) Minimum shell thicknesses are recommended for all steel penstocks to provide the rigidity needed during fabrication and handling. This minimum may be computed from the formula $T = D + 20/400$, where D equals the diameter in inches and T equals the minimum shell thickness in inches. A thinner shell may, in some cases, be used if proper stiffeners are provided during fabrication, handling, and installation.

(c) Welded joints should be butt-welds made under strict procedure control by qualified operators, and in accordance with the provisions of CW-05550, welded power penstock and surge tanks. All longitudinal seams should be examined radiographically in accordance with CW-05550.

(d) Completed penstocks with an operating head greater than 100 feet, should be hydrostatically tested with an internal pressure that will produce a hoop stress of 1.5 times the allowable stress. Penstocks with operating heads less than 100 feet should be pressure tested if they are unusually long, as may be the case of power tunnels or some conduits. Care should be taken when specifying test pressures to indicate where on the penstock the pressure is to be measured. This will ensure that the penstock is not overstressed during the test. If the entire penstock cannot be tested as a unit, individual sections are to be tested in the shop after they have been radiographed. The pressure should be applied three times, being increased and decreased slowly at the uniform rate. The test pressure should be held for a length of time sufficient for the inspection of all plates, joints, and connections for leaks or signs of distress. It is desirable that the test be performed when the pipe and water have a temperature of not less than 60 °F. The penstock should be vented at high points during filling to prevent formation of air pockets.

(e) Upon completion, and prior to insulation and painting, surge tanks should be tested by filling the tank with water to a point 1 foot from the top of the shell and maintaining this water level for not less than 24 hours, or such additional time as may be required to inspect all plates, joints, and connections for leaks or signs of distress. Preferred water and shell temperatures for the test should be not less than 60 °F.

(f) In long penstocks, a surge tank may be necessary to prevent the fluctuation of water-hammer flow from seriously interfering with turbine regulation (see 4-16(f)). Free-standing penstocks should be constructed so as to permit any leakage to drain to tailwater without pressurizing the surrounding regulating outlet conduit. Careful attention should be given to anchorage of the penstock against longitudinal thrust.

c. Power conduit linings. The function and many of the details of construction and erection for an integrally embedded steel liner are similar to a free-standing penstock; however, the loading conditions are different. The steel lining, concrete encasement, and if present, the surrounding rock act together to resist the pressures. EM 1110-2-2901 outlines in detail the loading conditions and allowable stresses for a conduit under embankments or rock. In both instances, external pressures must be accounted for as well as the internal pressures.

d. Water velocities and water hammer. The velocity of flow in penstocks depends largely upon turbine regulations but is seldom lower than 6 feet per second. In very high-head plants velocities as high as 30 feet per second have been used. For medium-head plants at maximum discharge, velocities of about 12 to 18 feet per second are typical. It should be noted that the allowable stresses for the components of the turbine spiral case, spiral case extension, valve, and valve extensions are different than those for the penstock. Refer to the guide specifications (CE-2201.01, CE-2201.02, and CE-2201.03, etc.) for these allowable stresses. The point of division between the penstock and spiral case/valve extension is customarily defined as the limit of supply for the turbine and/or valve manufacturer.

(1) Changes in the rate of flow in penstocks cause variations in pressure known as water hammer. These changes in flow rate can be caused by the turbine wicket gate motions due to power changes or load rejections, unit runaway, and closure of the emergency valve or gate. The magnitude of the pressure variation is dependent upon the length of penstock, the velocity of the

water, and the rate of change of the flow. When the turbine gates close due to a decrease in load, the pressure increases above the steady full load gradient. As the gate movement ceases, the gradient drops below that for steady full load, then fluctuates with diminishing amplitude between the maximum and minimum positions until the movement is damped out by friction. When an increase in load causes the turbine gates to open, the gradient first drops below that for steady full load, then fluctuates in a manner similar to that described for gate closure. The penstock must be designed at every point to withstand both the maximum and minimum pressure at that point as determined by the highest and lowest position of the water-hammer pressure gradient.

(2) The subject of water hammer is covered in *Hydroelectric Handbook* by Creager and Justin, *Handbook of Applied Hydraulics* by Davis, *Engineering Fluid Mechanics* by Jaeger, and *Waterhammer Analysis* by John Parmakian. Prior to development of plans and specifications, the hydraulic system should be modeled using a digital computer to simulate the various design conditions and configurations of the hydroelectric facility. The Corps has had a computer program (WHAMO) especially developed to simulate water hammer and mass oscillation in hydro-power and pumping facilities. This program or one equal to it should be used for this purpose. The Hydroelectric Design Center should be consulted prior to usage of WHAMO.

e. Bends, wyes, and tees. The distance from an elbow or bend in a penstock to the turbine inlet should be as great as the layout will permit in order that disturbances in the flow at the bend will not affect turbine performance. If a butterfly valve is used, its center line should be at least 3 penstock diameters upstream from the center line of the unit. The penstock must be anchored at bends to withstand the centrifugal forces of the water as it changes direction. Anchorages are usually blocks of mass concrete encasing the pipe. Wyes and tees involve internal pressures on noncircular sections and require special design. Often the entire wye branch or tee is encased in reinforced concrete, sometimes with embedded steel girders and tie rods, to prevent deformation and concentration of stresses in the shell. An example is the tee at a surge tank riser. Stress analysis of wyes and tees can be done as outlined in “Design of Wye Branches for Steel Pipe” published in the June 1955 edition of *Journal of The American Water Works Association*. For large structures or unusual configuration, a finite element analysis may be necessary.

f. Surge tanks and stability. For isochronous (isolated from the power grid) operation, a minimum ratio of water-starting time to mechanical starting time is required for stability. Usage of a surge tank (which decreases water starting time) or a flywheel (increases mechanical starting time) may be employed. Surge tanks also moderate water-hammer pressures. In long penstocks the fluctuation of water-hammer pressure may seriously interfere with turbine regulation unless relief is provided. For this purpose, a surge tank is generally used at the lower end of a penstock longer than about 400 feet. For simple surge tanks, the minimum area is usually 50 percent larger than the Thoma area. Isochronous operation capability should be provided for all but the smallest units. Surge tanks are of three basic types: simple, restricted-orifices, and differential. Also for underground stations where the rock is suitable, a surge chamber (accumulator) can be employed. A discussion of the advantages and disadvantages of each type as well as an outline of design procedures,is contained in Chapter 35 of *Hydroelectric Handbook* by Creager and Justin. *Waterhammer Analysis* by Parmakian also covers solutions of simple surge tanks. The WHAMO program is also capable of medeling all types of surge tanks as well as predicting hydraulic instability. It is recommended that the Hydroelectric Design Center or an engineer who has a successful record in surge tank design be retained to analyze the flow regulation problem and design the tank at any power project where long penstocks are to be used and isochronous unit operation is a requirement.

4-17. Switchyard Structures

The most suitable and economical general arrangement and design of outdoor high-voltage switchyards should be based on consideration of the scheme of high-voltage switching employed, the voltage and capacity of the main buses and transmission lines, the number of generator or transformer and transmission line bays required, the location of the main power transformers, the direction of transmission lines leaving the yard, and size and topography of the space available.

a. The switchyard should be arranged to provide adequate space for the safe movement of maintenance equipment and for the future movement of circuit breakers and other major equipment into position without de-energizing existing buses and equipment. A chain link woven wire fence approximately 7 feet with lockable gates should be provided to enclose the entire switchyard.

b. An arrangement using high truss-type structures and either strain or rigid-type buses required a minimum of ground area and is generally used for yards rated 161 kv and below. An arrangement using low flat-type structures with rigid buses is generally used for yards 230 kv and above and may also be used for lower-voltage yards where adequate space is available. This design utilizes separate A-frame structures for dead-ending the transmission lines and individual lightweight structures for supporting the buses, disconnects and other equipment. This arrangement is considered the most reliable and all equipment is easily accessible for inspection and maintenance.

c. The switchyard structures and transmission line take-off towers have special requirements in regard to loading, rigidity, resistance to shock, installation, and maintenance. Standards for the design of switchyard structures to meet these special requirements have been developed on the basis of long experience of the power industry and are summarized in NEMA Publication SG 67, “Power Switching Equipment.”

d. The switchyard structures should be designed for the initial power installation but with provision for expansion as additional generating units and transmission lines are installed in the future.

4-18. Reinforcing Steel

Reinforcement should be designed using the requirements set forth in the latest edition of ACI 318 “Building Code Requirements for Reinforced Concrete,” and as amended by EM 1110-2-2104 “Strength Design for Reinforced Concrete Hydraulic Structures.” Guide specification CW-03210 “Steel Bars, Welded Wire Fabric and Accessories for Concrete Reinforcement” provides the necessary details for tests, cutting, bending, and splicing of reinforcement.

4-19. Encasement of Structural Steel

When the framing of the powerhouse is structural steel, the members shall not be encased, except for certain locations where appearance is a factor, such as office space and lobby, or where a fire hazard exists.

4-20. Retaining Walls

Walls subject to earth pressure, such as tailrace walls and foundation walls at the shore end of the powerhouse, may be of the gravity, semi-gravity or cantilever type, depending upon economy, and should be founded on solid rock wherever possible. Where sound rocks rises above the bottom of the tailrace at the shore side, excavation may be saved by anchoring a concrete facing to the rock and building the gravity wall above.

4-21. Area Drainage

Roof drains should be provided with basket-type strainers and should be connected to interior leaders discharging into the headwater or the tailrace. All floors should have flush drains to carry wash water, seepage, and possible leakage from tanks to the station sump or, if the area drained is well above maximum tailwater, to the tailrace.

a. Angles and abrupt bends in drain lines should be avoided insofar as possible, and cleanouts should be provided where necessary to facilitate clearing the pipes.

b. Floors should be sloped so that drains are well removed from electrical equipment, and particular attention should be paid to all details to avoid damage to such equipment caused by leaks or clogged drains.

c. In cold climates drain piping must be located where temperatures will not drop below freezing or must be properly insulated. Outlets should be well below tailwater to prevent formation of ice at the discharge.

4-22. Chamfers, Grooves, and Rustications

Exterior square corners are undesirable in concrete construction because of their tendency to break removal of forms or as a result of weathering or impact. Chamfers are usually formed on all exposed corners, but particular attention should be paid to this detail on the exterior walls. Chamfers of ample size should be provided at the ends of monoliths, forming V-grooves at the contraction joints. Horizontal V-grooves, or rustications are sometimes used for architectural reasons and lift heights should then be planned so that the rustication will occur at the horizontal joints.