# Appendix B12

# Preliminary Design Criteria

# 13-20-TM_Preliminary Design Criteria

SUSITNA-WATANA HYDRO

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**NTP 13**
**Technical Memorandum No. 4**
**v2.0**
**UPDATED DRAFT**

# PRELIMINARY DESIGN CRITERIA
# 2014

**AEA11-022**


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# TABLE OF CONTENTS

1. GENERAL INTRODUCTION ..... 1
1.1. Preliminary Design Criteria and Study Methodologies ..... 1
1.2. Overall Project Description ..... 1
2. STANDARDS ..... 1
3. RESERVOIR ..... 2
4. SITE TOPOGRAPHY AND HYDROLOGY ..... 2
4.1. Topographical Data ..... 2
4.2. Hydrological Data ..... 2
4.3. Reservoir Data ..... 2
4.4. Tailwater Data ..... 3
4.5. Design Floods ..... 4
5. HYDRAULIC DESIGN ..... 5
5.1. Power Waterways ..... 5
5.1.1. Power Intake ..... 5
5.1.2. Gates and Stoplogs ..... 6
5.1.3. Power Conduit ..... 7
5.2. Spillway ..... 10
5.2.1. General ..... 10
5.2.2. Spillway Location ..... 10
5.2.3. Discharge Capacity ..... 11
5.2.4. Crest Shape ..... 11
5.2.5. Discharge Characteristics ..... 13
5.2.6. Piers and Abutments ..... 14
5.2.7. Gates ..... 14
5.2.8. Chute and Flip Bucket ..... 15
5.3. Low Level Outlets ..... 17
5.4. Emergency Release Facilities ..... 18
6. SITE GEOLOGY AND GEOTECHNICAL DESIGN ..... 19

6.1. General ..... 19
6.2. Site Geology ..... 19
6.3. Previous Exploration Programs ..... 19
6.4. Hydrogeologic Conditions ..... 20
6.5. Geotechnical Parameters ..... 20
6.5.1. Intact Rock Properties ..... 20
6.5.2. Rock Mass Strength and Deformation ..... 21
6.5.3. Rock Mass Poisson's Ratio ..... 21
6.5.4. Shear Strength of Rock Discontinuities ..... 21
6.5.5. Rock Mass Permeability ..... 22
6.5.6. Bedrock Concrete Interface ..... 23
6.5.7. Overburden ..... 23
6.5.8. Engineered Fills ..... 25
7. SLOPE STABILITY ..... 25
7.1. General ..... 25
7.2. Overburden / Fill Slope Stability Analysis ..... 26
7.3. Rock Slope Stability Analysis and Support ..... 26
7.4. Seismic Loads for Stability Analysis ..... 27
7.5. Numerical Modeling ..... 28
7.6. Phreatic Surfaces for Stability Analyses ..... 28
7.7. Slopes for Excavations and Fills ..... 28
7.7.1. Bedrock Excavations ..... 28
7.7.2. Overburden Excavations ..... 29
7.7.3. Compacted Fills ..... 29
7.8. Minimum Factors of Safety for Excavations and Fills ..... 29
7.9. Slope Protection ..... 30
7.10. Seepage Analysis ..... 30
8. FOUNDATIONS – DAMS AND STRUCTURES ..... 30
8.1. Bearing Capacity of Foundations on Rock ..... 30
8.2. Bearing Capacity of Foundations on Soils ..... 31
8.3. Lateral Earth Pressures ..... 31

8.4. Settlement ....32
8.5. Dam Foundation Preparation ....32
8.6. Dam Foundation Grouting ....33
8.7. Dam Foundation Drainage ....34
8.8. Dam Foundation and Abutments Sliding Stability ....34
8.9. Plunge Pool Scour ....35
8.10. Lateral Earth Pressures ....38
8.11. Settlement ....38
9. UNDERGROUND EXCAVATIONS ....38
9.1. Rock Support Design ....38
9.2. Tunnel Dewatering Estimates and Design ....40
10. SEISMICITY ....41
10.1. Seismicity Studies ....41
10.2. Seismic Criteria ....41
11. GENERAL STRUCTURAL CRITERIA ....44
11.1. Structural Analysis and Design ....44
11.2. Material Properties for Design ....44
11.2.1. Water ....44
11.2.2. Structural Concrete ....44
11.2.3. Steel Reinforcement ....44
11.3. Design Loads ....45
11.3.1. Dead Loads ....45
11.3.2. Live Loads ....46
11.3.3. Backfill Loads ....47
11.3.4. Snow and Ice Loads ....47
11.3.5. Crane Loads ....48
11.3.6. Spillway Deck Loads ....48
11.3.7. Hydraulic Loads ....48
11.3.8. Uplift ....48
11.3.9. Seismic Loads ....50
11.3.10. Temperature and Thermal Loads ....50

11.3.11. Wind Load ....51
11.4. Load Cases ....51
11.5. Minimum Safety Factors and Allowable Stresses ....52
11.5.1. Safety Factors against Overturning ....52
11.5.2. Safety Factors against Sliding ....53
11.5.3. Safety Factors against Flotation ....54
11.5.4. Allowable Stresses ....54
12. RIVER DIVERSION ....54
12.1. General ....54
12.2. River Diversion ....55
13. RCC DAM ....55
13.1. Design of Dam ....55
13.1.1. Type of Structure ....55
13.1.2. Dam Geometry and Layout ....56
13.2. Material Properties ....56
13.2.1. Water ....57
13.2.2. RCC ....57
13.2.3. Foundation Rock ....57
13.3. Design Loads ....57
13.3.1. Dead Load ....58
13.3.2. External Hydrostatic Loads ....58
13.3.3. Hydrodynamic Load ....58
13.3.4. Internal Hydrostatic Loads (Uplift) ....58
13.3.5. Earthquake Loads ....58
13.3.6. Ice Loads ....59
13.4. Load Combinations ....59
13.5. Structural Stability Analysis ....60
13.5.1. Static ....60
13.5.2. Dynamic ....61
13.5.3. Thermal Analysis ....61
14. POWERHOUSE ....61

15. ROADS AND ACCESS INFRASTRUCTURE ..... 62
15.1. ARRC Offloading Facilities ..... 62
15.2. Access Road to the Site ..... 63
15.3. Access Around the Site ..... 63
15.4. Permanent Bridge ..... 65
16. POWER FACILITIES ..... 65
17. SITE INFRASTRUCTURE ..... 65

## Table of Figures

Figure 4-1. Reservoir Elevation, Area and Volume Curves ..... 3
Figure 4-2. Tailwater Rating Curve ..... 4
Figure 5-1. Three-arc Configuration ..... 12
Figure 8-1. Erodibility of Earth and Rock Materials by Hydraulic Jets (Annandale 1995) ..... 37

## Table of Tables

Table 4-1. Tailwater Rating Curve ..... 3
Table 4-2. Estimated Flood Frequency ..... 4
Table 5-1. Origin of the Radii and Transition Points ..... 12
Table 5-2. Reservoir Evacuation Criteria ..... 18
Table 7-1. Recommended Slopes for Different Soils ..... 29
Table 10-1. Preliminary Seismic Design Parameters for Susitna Structures and Excavations ..... 43
Table 11-1. Live Loads ..... 46
Table 11-2. USACE Factors of Safety for Concrete Structures ..... 51
Table 11-3. USACE Load Cases for Concrete Structures ..... 52
Table 13-1. Stability and Stress Criteria ..... 60
Table 14-1. Estimated Tailwater Levels at Various Discharge Conditions ..... 62
Table 15-1. Proposed Design Criteria for Access Road to the Site ..... 63
Table 15-2. Proposed Preliminary Design Criteria for Roads within Project Works ..... 64

# 1. GENERAL INTRODUCTION

## 1.1. Preliminary Design Criteria and Study Methodologies

This document provides suggested approaches, methodology, design parameters, and criteria for hydraulic, civil, structural, geotechnical, and seismic design elements for the Susitna-Watana Hydroelectric Project. Power facilities and the associated electrical and mechanical balance of plant are described in the Draft Engineering Feasibility Report. The preliminary designs prepared for the various project features thus far utilize many of these criteria, and will need to be updated during the next stages of preliminary and detailed design engineering for the project. Later stages of engineering design may use much of the same design criteria defined herein, developed further.

At time of issue of this Technical Memorandum, limited information was available from the recent site investigation programs conducted during the summer and fall of 2014. In the absence of confirmatory information from the site investigations, the parameters used in the studies and calculations pertaining to project feasibility should be conservative.

This document must be revised carefully, including incorporation of the Owners Requirements and when available – geotechnical information obtained from ongoing studies. It should not be used for detailed design until such revision, checking and validation has been carried out by the Engineer of Record prior to commencing detailed design.

## 1.2. Overall Project Description

A detailed description of the project, as currently proposed, is included in Section 10 of the 2014 Draft Engineering Feasibility Report.

# 2. STANDARDS

The hydraulic, civil, structural, geotechnical and seismic design will be based on the applicable portions of the codes, standards, methodology and publications referenced. The latest issues will apply, and equivalent standards from other authorities may be used where demonstrably equivalent. In the event of conflict, the most conservative standards will apply.

# 3. RESERVOIR

The normal reservoir operating limits are:

| | |
|---|---|
| Probable Maximum Flood level (PMF) | 2064.5 ft. |
| Normal Maximum Operating Level (NMOL) | 2050.0 ft. |
| Minimum Operating Level (MOL) | 1850.0 ft. |

The dam crest will include a concrete parapet wall of a minimum height of 3 ft. sufficient to contain wave and wind run up without overtopping, and robust enough to prevent a truck from entering the reservoir from the dam crest.

# 4. SITE TOPOGRAPHY AND HYDROLOGY

## 4.1. Topographical Data

The topography of the project site and reservoir is based on aerial survey mapping. The survey data is based on the following properties:

| | |
|---|---|
| NGS State Plane Coordinates: | Zone 5004 (Alaska Zone 4) |
| Datum: | North American Datum 1983 (NAD83) |
| | North American Vertical Datum 1988 (NAVD88) |

Any data in other coordinate systems should be converted before use.

## 4.2. Hydrological Data

The hydrological background to the area and project is included within the Draft Engineering Feasibility Report.

## 4.3. Reservoir Data

Reservoir elevation, area and storage volume curves for Watana Reservoir are given below:

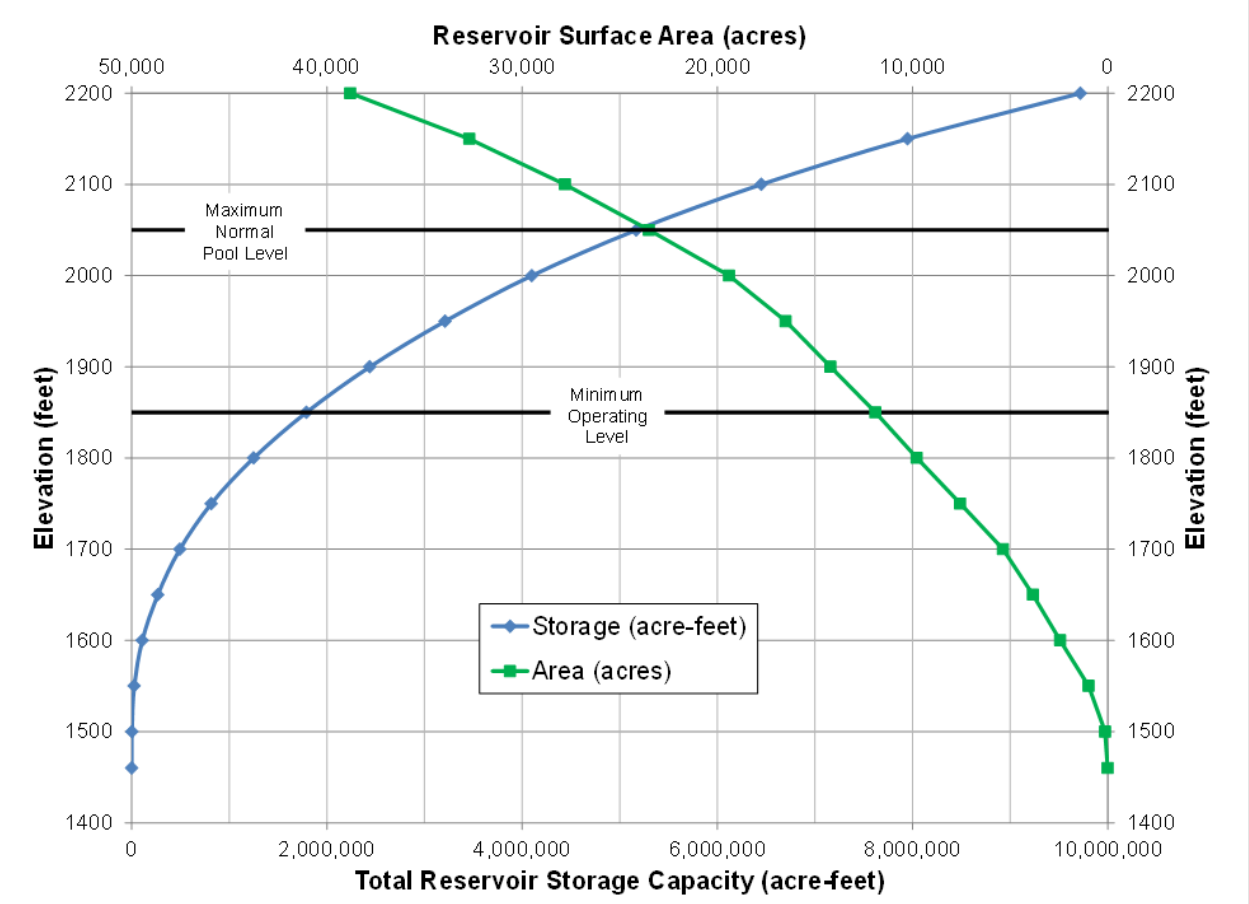


**Figure 4-1. Reservoir Elevation, Area and Volume Curves**

## 4.4. Tailwater Data

The tailwater rating curve at Watana dam is summarized below. The curve is based on cross-section data surveyed in 2014. The vertical datum for the tailwater rating curve is NAVD 88, Geoid09.

**Table 4-1. Tailwater Rating Curve**

| Elevation | Discharge (cfs) |
|---|---|
| 1484.0 | 150,000 |
| 1475.5 | 80,000 |
| 1468.8 | 40,000 |
| 1464.6 | 20,000 |
| 1461.1 | 8,000 |
| 1459.4 | 4,000 |

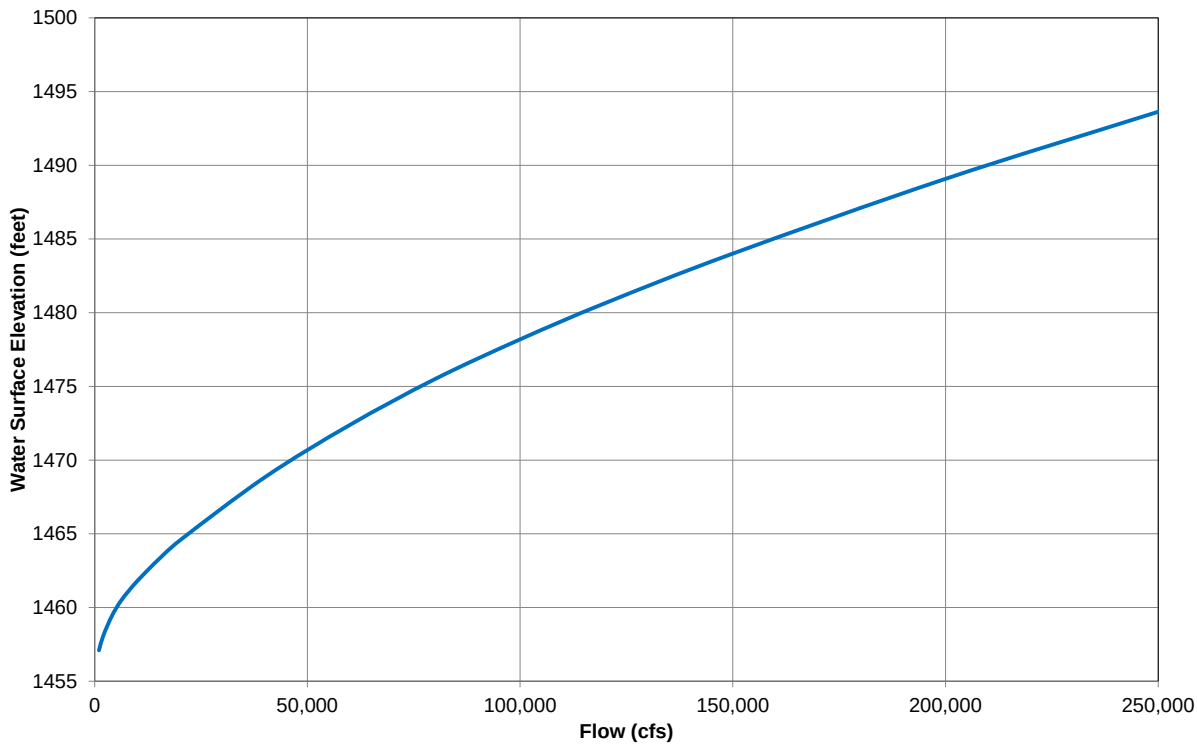


**Figure 4-2. Tailwater Rating Curve**

## 4.5. Design Floods

The estimated flood frequency at the Watana Dam site is as follows:

**Table 4-2. Estimated Flood Frequency**

| Return Period (Years) | Peak Flow (cfs) |
|---|---|
| 2 | 38,500 |
| 5 | 50,500 |
| 10 | 59,200 |
| 25 | 71,300 |
| 50 | 80,800 |
| 100 | 91,300 |
| 500 | 116,300 |
| 1,000 | 128,400 |
| PMF | 310,000 |

# 5. HYDRAULIC DESIGN

## 5.1. Power Waterways

### 5.1.1. Power Intake

The project will include four separate intakes, located on the upstream face of the RCC dam, one for each of the three hydroelectric generating units and one spare, available for a future unit. The design will incorporate moveable shutters at five levels so that draw off may be selected at appropriate levels.

Within practical limits, the future intake should be designed for unit flows appropriate to the largest unit that can be accommodated by the future unit bay.

The three intake structure openings for the initially installed units will be protected by trashracks. A floating heated boom will be provided to prevent ice from forming on the trashracks.

The power intakes will be designed with special attention to approach and entrance flow conditions. The hydraulic design of approach conditions will provide for uniform, symmetrical approach flow. The internal geometry of the intakes will be selected to minimize hydraulic losses to the extent practical.

The power intakes will be set low enough to provide sufficient submergence at all operating reservoir levels to prevent unacceptable vortex formations at maximum discharge.

The key criteria applicable will be as follows:

***Intake Characteristics***

For each intake, at each of five elevations, assuming that the only one intake level is used at any time, the net intake area will be determined by the limitation of gross velocity to 5 ft. per second maximum, unless negotiated fish protection criteria call for lower velocities.

Energy losses through the intake will be limited to 0.5 ft. or less and will be derived from:

$$h_L = K_t \frac{V_n^2}{2g}$$

$$K_t = 1.45 - 0.45\frac{A_n}{A_g} - \left(\frac{A_n}{A_g}\right)^2$$

Where:

$V_n$ = velocity based on the net area

$A_n$ = net area of trashrack and support structure

$A_g$ = gross area of trashrack and support structure

Instrumentation will be provided at the intake to measure differential head, with a meter or display in the control room or operations center so that operating personnel can monitor and detect a condition of excessive head loss.

***Trashracks***

Bar dimensions and spacing will be sized to limit vibration during peak flow, and for fish protection criteria.

***Submergence***

Submergence will be calculated to avoid the possibility of vortex formation sufficient to draw in air or debris. Assuming symmetrical approach conditions, required minimum submergence will be calculated in accordance with the following empirical relationship:

$$\frac{S}{d} = C'\left(\frac{V}{\sqrt{gd}}\right)$$

Where:

$S$ = submergence required to prevent vortex formation

$d$ = penstock diameter

$V$ = average intake velocity

$C'$ is an empirical dimensionless coefficient of 1.7, representing symmetrical approach flow

Trashracks and shutters will be removable and – if feasible – will be serviced by the same gantry crane(s) supplied for servicing of the intake gates and stoplogs.

### 5.1.2. Gates and Stoplogs

Hydraulically operated intake gates capable of closure under turbine runaway flow will be installed in slots downstream of the shutters and trashracks. Vertical slots for stoplogs will be provided between the intake gates and the shutters.

The stoplogs will incorporate a bypass system so that the downstream side may be flooded to enable the stoplogs to be raised under balanced conditions.

### 5.1.3. Power Conduit

The power conduit will have a reinforced concrete section and a steel section encased in concrete.

***Optimization***

Optimization of the power conduit diameter will be based on an evaluation of the construction cost in comparison with total head losses and the corresponding energy loss. Maximum flow velocity will be limited to 18 ft./sec in concrete sections, and 30 ft./sec in steel sections.

For feasibility, the diameter of the penstock will be determined according to the following empirical formulae:

Gulliver (1991) $D = 9.554\left(P^{0.43}/H^{0.57}\right)$

Sakaria (1979) $D = 13.83\left(P^{0.43}/H^{0.65}\right)$

Fahlbusch (1987) $D = 1.12\left(Q^{0.45}/H^{0.12}\right)$

Where:

D = economical penstock diameter (m)

$P$ = rated turbine capacity (MW)

$H$ = rated net head (m)

$Q$ = rated flow (m3/sec)

Also:

Warnick (1984) $D = 4.44\left(P^{0.43}/H^{0.63}\right)$

Where:

D = economical penstock diameter (feet)

$P$ = rated turbine capacity (horsepower);

$H$ = rated net head (feet)

***Operating Stability***

Following the derivation of the economical diameter, the operating stability of the generator-penstock will be derived. One of the following two criteria will be used in determining the operational stability:

Criterion 1: $T_m/T_w > 3.0$ (Hadley 1970)

Criterion 2: $T_m/T_w^2 > 2.0$ (USBR 1976)

The desired ratio will be achieved – if necessary – by increasing the unit inertia or increasing the penstock diameter. The mechanical starting ($T_m$) time will be determined using the formula (metric units):

$$T_m = \left( {GD^2N}/{36 * 10^4 P} \right)$$

The water column starting time ($T_w$) shall be determined using the formula:

$$T_w = {\Sigma(LV)}/{gH}$$

***Head Losses***

For head loss calculations relating to power and energy estimates, optimization and conduit design, the following head loss equation for form changes will be used:

$$h_L = K \frac{V^2}{2g}$$

Where:

$V$ = flow velocity

$g$ = acceleration due to gravity

$K$ = form loss coefficient

Dimensionless values of $K$ may be estimated by the geometry for each type of component. Factors found in USACE EM 1110-2-1602, plates C-8 through C-14 and from Davis Handbook, Section 2 will be used.

The intake geometry will be determined to result in an intake loss coefficient of 0.1 or less (per Table 3.8 of Guidelines for Design of Intakes for Hydroelectric Plants (ASCE 1995).

The penstock friction head loss will be calculated using Manning's equation. The following "n" values will be used:

Concrete: n = 0.013

Steel: n = 0.012

***Structural Design***

The design static head on the penstocks will be 777 ft., to accommodate any future dam raise resulting in a normal maximum water level of El. 2185 ft. msl (based on routed PMF level). For the purposes of feasibility design an allowance of 40% should be made on the static head to allow for transient pressures, but for final design a transient study should be made including sudden gate closure and turbine runaway (which could be more critical than gate closure) to determine maximum transient pressure. Preliminary computations estimate that the selected turbines, under runaway, may choke the flow by up to 40% leading to the critical transient pressure rises.

For handling, the minimum steel thickness will be calculated using the following formula:

$$t_{min} = {}^{D + 20}/_{400}$$

Where:

$t_{min}$ = Minimum Shell thickness (in)

$D$ = Penstock diameter

For the normal condition, (static plus transient pressure rise), the minimum plate thickness is:

$$t = {}^{PR}/_{S_A}$$

Where:

$t$ = Shell thickness (in)

$P$ = Design pressure (lbs./in$^2$)

$R$ = Penstock inside radius (in)

$S_A$ = specified allowable stress (lbs./in$^2$)

## 5.2. Spillway

### 5.2.1. General

The spillway will consist of a four-gated chute integrated into the body of the main dam, with a central wall that shall not be overtopped by a 60,000 cfs release on one half of the chute. The chute will terminate with a flip-bucket, discharging flow into a plunge-pool for energy dissipation.

The hydraulic design of the spillway will be based on the following three references:

- USACE EM-1110-2-1603, Hydraulic Design of Spillways;
- Davis Handbook of Applied Hydraulics, Fourth Edition, by Zipparro and Hasen 1993, Chapter 16, Overfall Spillways; and,
- MWH Best Practice Dams and Hydropower Guideline, Hydraulic Design of Spillways.

### 5.2.2. Spillway Location

The spillway will be located on the right abutment, on the crest of the dam and the downstream face. The spillway orientation will be straight, approximating a radial direction and extending downstream to align the plunging jet, if possible, along the direction of tailrace flow in the river channel.

The spillway will be located such that topography within the reservoir does not cause asymmetrical flow to the ogee crest. Asymmetrical flow is avoided if there is at least 1.5 times the total width of the spillway upstream of the crest without topography (vertical or horizontal) that would affect flow.

Rock may need to be excavated from the area immediately upstream of the spillway in order to maintain approach velocity below 4 to 6 ft./sec.

### 5.2.3. Discharge Capacity

Discharge capacity requirements are as follows:

1. The reservoir level resulting from the occurrence of the PMF – with the initial condition reservoir level of 2050 ft., and with all gates operating – is not to exceed a level of 2064.5 ft. This assumes that all discharge valves of the low-level outlet are fully discharging.

2. With one gate inoperable, and an initial condition reservoir level of 2050 ft., the spillway and low-level outlets shall be able to pass a 1:10,000 yr. flood without overtopping the dam.

3. The estimated spillway discharge capacity (not including low-level outlets) to satisfy the above PMF criteria is approximately 250,000 cfs.

### 5.2.4. Crest Shape

The discharge section of the spillway will be an ogee shaped crest to encompass the following criteria:

1. The ratio of design head to maximum head shall be about 0.75.

2. Consideration must be given to cavitation potential at the ogee crest. The minimum pressure allowable for design purposes will be 20 feet of water below sea-level atmospheric pressure, taking into account the altitude of the site (Zipparro and Hazen 1993, pg. 16.6-16.8).

3. The ratio of approach depth to design head shall be at least 2.0.

4. The upstream quadrant will comprise a three- arc configuration in accordance with the following sketch:

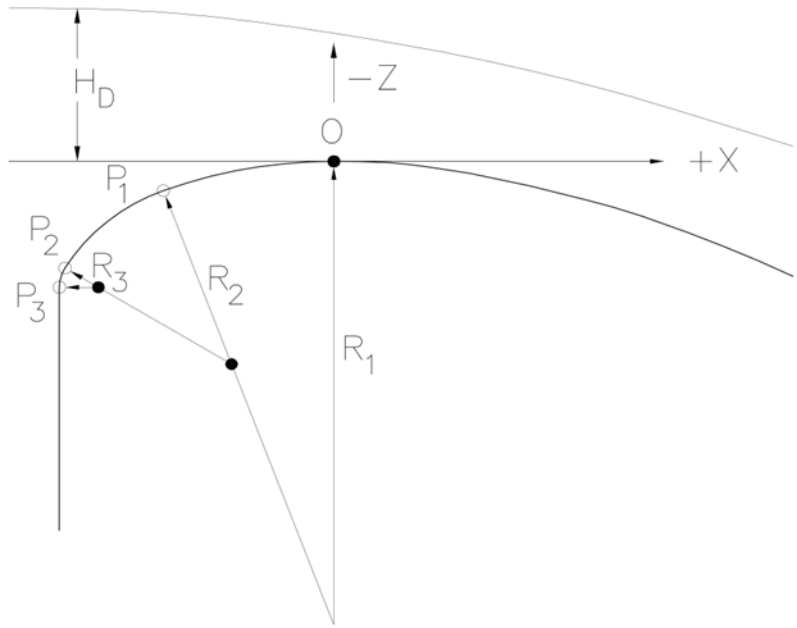


**Figure 5-1. Three-arc Configuration**

The shape will be described in accordance with the following:

- $R_1/H_d = 0.5$
- $R_2/H_d = 0.2$
- $R_3/H_d = 0.04$

The origin of the radii, and the transition points will be in accordance with the following table:

**Table 5-1. Origin of the Radii and Transition Points**

| Point | 01 | 02 | 03 | P1 | P2 | P3 |
|---|---|---|---|---|---|---|
| $X/H_d$ | 0.000 | -0.105 | -0.242 | -0.175 | -0.276 | -0.2818 |
| $Y/H_d$ | 0.500 | 0.219 | 0.136 | 0.032 | 0.115 | 0.1360 |

5. The downstream quadrant, when the ratio of the approach depth to design head is 2.0 or more, will be described by the following equation:

$$X^n = KH_d^{n-1}\,Y$$

Where:

$X$ = horizontal distance, positive downstream from the apex (ft.)

$Y$ = vertical distance, positive downward (ft.)

$n$ = variable, usually set to 1.85

$K$ = variable dependent on $P/H_d$ – 2 for a deep approach (item 2 above)

6. The position of the gate lip will be between 0.1 to $0.3H_d$ downstream of the crest location to ensure positive pressure.

### 5.2.5. Discharge Characteristics

It is proposed that a four-bay spillway with radial gates will be constructed. Spillway rating curves for both partially and fully open gates will be developed based on a detailed calculation procedure presented in the following documents:

- U.S. Army Corps of Engineers, Hydraulic Design of Spillways, EM 1110-2-1603, January 1990; and,
- U.S. Army Corps of Engineers, Hydraulic Design Criteria, Section 311, Tainter Gates on Spillway Crests, Discharge Coefficients, March 1958.

The discharge $Q$ over an uncontrolled (gates fully open) overflow structure can be expressed as:

$$Q = CL_eH_e^{1.5}$$

Where:

$L_e$ = Effective length of crest

$H_e$ = Total energy head

$C$ = Coefficient of discharge dependent on:

- $P/H_d$ and relative head on the crest $H_e/H_d$,
- The slope of upstream slope face,
- Crest submergence and downstream condition,
- Site-specific factors such as flow angularity resulting from complex approach flow geometry or unusually shaped piers,
- Crest submergence and downstream condition.

### 5.2.6. Piers and Abutments

Piers are included for the gated spillway as a support for a bridge over the spillway. Plate 3-6 through Plate 3-8 of the USACE EM 1110-2-1603 can be used for design of piers.

Crest piers and abutments cause contraction of the flow, reduction in the effective spillway crest length and ultimately a reduction in the spillway discharge. The following relationship applies:

$$L_e = L - 2\left(NK_p + K_a\right)H_e$$

Where:

$L$ = Total length of crest

$N$ = Number of piers

$K_p$ = Pier contraction coefficient, shown on Plate 3-6 through Plate 3-8 of USACE EM 1110-2-1603

$K_a$ = Abutment contraction coefficient shown on Plate 3-11 and Plate 3-12 of USACE EM 1110-2-1603.

The above equation can be used for a gated spillway when the gate is fully open. For partial openings, the orifice flow equation should be used.

### 5.2.7. Gates

The spillway gates will be radial gates capable of being raised out of the flow at higher discharges.

Unit discharge at the flip bucket will be no more than 2000 $ft^3/ft$. The chute may contract for optimum flow characteristics.

Stoplogs or bulkheads will be provided so that one spillway gate may be dewatered, with the reservoir at elevation 2057.5 ft.

Closed spillway gates will contain a reservoir surcharge of 7.5 ft., i.e., a water level of 2057.5 ft., with a wave deflector above.

At the top of each spillway gate, a flap gate will be included, at least 25 ft. wide and 4 ft. high, which can be operated independent of the gate. The flap gates shall be able to be lowered to flush trash which accumulates in front of the spillway gates. They shall be designed to ensure that discharge and trash do not adversely affect the spillway gate arms or the main gate hoist structure.

The gate seal wipe plates will be heated to prevent ice formation.

### 5.2.8. Chute and Flip Bucket

***Chute***

The spillway chute may be of constant width, or there may be a transition between the width at the crest gate structure and the flip bucket. The following criteria will be used for the transition section:

- Convergence must be symmetrical;
- The transition should be close to the control section where velocities will be lowest, and in any case should not extend beyond the section of the chute where the Froude number is above 1.5;
- The transition should be as smooth as possible and be proportioned to maintain supercritical flow to the extent possible; and,
- If possible a "funnel" contraction should be used and the tangent of the angle of contraction less than $1/(3F_r)$, where the Froude number is calculated based on average depth and velocity in the transition zone. In any case, the contraction angle should be less than 10°. Figure 1 of the MWH Guideline, Hydraulic Design of Spillways, provides general guidance on spillway chute shapes.

The side walls of the chute should be a minimum of 2 ft. higher than the design water level – including aeration. However, it is preferable to use the more conservative USBR (1977) recommendation of freeboard:

$$Freeboard\ (ft) = 2.0 + 0.025VD^{\frac{1}{3}}$$

Where:

$V$ = mean velocity (ft/sec)

$D$ = mean depth (ft)

The general references for the determination of cavitation potential and aeration requirements are USBR Engineering Monograph No. 42, Cavitation in Chutes and Spillways, and Chapter 16 of Davis' Handbook of Applied Hydrology.

A guide to the necessity for aeration and or special construction can be determined from the cavitation index $\sigma$ (SI units):

$$\sigma = \frac{P + P_v + P_a}{\left(V^2/2\right)\rho_w}$$

Where:

$P$ = Static pressure, kPa

$P_a$ = Atmospheric pressure (sea level) = 101.00 kPa

$P_v$ = vapor pressure of water (20°C) = 2.33 kPa

$\rho_w$ = Unit weight of water (20°C) = 998.23 kg/m$^3$

$V$ = flow velocity, m/s

If the cavitation index is 3, or greater, no cavitation is possible. If the cavitation index is 0.2 or less, aeration or geometry changes are necessary. Aeration of the chute must be provided if the water velocity exceeds 100 ft./sec (based on the friction of new concrete) and thenceforth maintained at 10% (at chute floor) throughout the chute.

***Flip Bucket***

The arrangement of the flip bucket will be configured to deliver the jet to the most appropriate part of the river. The flip bucket lip shall be at least 20 ft. above maximum tailwater.

The bucket radius will be calculated according to the following equation (USBR 1977):

$$R = {}^{2qv}/_{p}$$

Where:

$R$ = minimum radius of curvature (ft) – not less than 10 times water depth for high velocity flow in high spillways

$q$ = unit discharge (cfs/ft)

$v$ = velocity (ft/sec)

$p$ = normal dynamic pressure on the floor (lb/ft$^2$) which should not exceed 500 (lb/ft$^2$)

The jet trajectory (origin at end of lip; y is vertical (ft.) and x is horizontal)) is determined by (USBR 1977):

$$y = xtan\theta - {}^{x^2}/_{4K(d+h_v)cos^2\theta}$$

Where:

$\theta$ = angle of the edge of the lip with the horizontal (degrees)

$K$ = a factor, which should be 0.9 to allow for air resistance

$h_v$ = velocity head at the bucket (ft.)

$d$ = depth of flow at the bucket (ft.)

The horizontal range of the jet at the level of the lip can be calculated as follows:

$$x = 2K(d + h_v)sin2\theta$$

***Plunge Pool***

The plunge pool will develop through the erosion of the river gravels and the underlying bedrock. During detailed design, a hydraulic model may be used to verify the potential erosion based on data obtained from geological site investigations of the bedrock at the proposed plunge pool location.

## 5.3. Low Level Outlets

Low-level outlets will be provided to allow releases to be made from the reservoir during filling and to provide a means for making non-power releases from the reservoir.

The capacity of the low level outlets will be selected to meet the requirements as determined during environmental studies and negotiations to provide reservoir drawdown capability.

The concept phase design used in the 1980s has been incorporated in the layout at this time and includes two low-level outlet conduits. The conduits will be sized for normal maximum water level in conjunction with full powerhouse flow to pass the 50-year flood without opening any spillway gates.

Each low level outlet is expected to divide in four and discharge through four Howell Bunger (or similar) fixed-cone valves mounted below the spillway flip bucket. Discharge will be directed towards the plunge pool.

The intakes for the low level outlets shall be set at the same level as the power intake, and shall be of similar design, incorporating trash racks, isolating gates and stop logs, but not shutters or heated ice boom. If possible, they shall be placed so that the same gantry crane used for the power intakes will function for the low level outlet intakes.

The intakes and the low level outlet steel pipes shall be designed using the same hydraulic and structural criteria as those used for the power conveyances.

The design of the low-level outlets will adopt applicable criteria stated in EM-1110-2-1602 - Hydraulic Design of Reservoir Outlet Structures, or other criteria as appropriate.

## 5.4. Emergency Release Facilities

The emergency release facilities would be used in conjunction with the spillway, fixed-cone valves in the low-level outlet conduits, and power waterways to release (according to criteria) up to 90% of the reservoir volume in 4 months. This proposed timescale is in compliance with the reservoir drawdown criteria contained in the U.S. Army Corps of Engineers publication ER 1110-2-50 "Low Level Discharge Facilities for Drawdown of Impoundments", dated August 22, 1975, but it is recognized that this criteria may have to be waived.

The reservoir is assumed to be at the full supply storage level at the start of the drawdown period. Additional reservoir drawdown criteria relate to the average reservoir inflow during the drawdown period. The drawdown period inflow is assumed to be the average flow of the highest consecutive four month period of inflow, which would be during the May through October period for Watana Reservoir. Reservoir evacuation levels and volumes are shown below in Table 5-2.

The emergency release system will be located in the diversion tunnel. The tunnel would be locally deepened to accommodate the emergency release outlet gates and expansion (stilling) chamber. These facilities would be used to pass minimum flows during initial filling of the lower levels of the reservoir in addition to emergency draining of the reservoir.

**Table 5-2. Reservoir Evacuation Criteria**

| Parameter | Value |
|---|---|
| Initial reservoir level (feet) | 2050 |
| Initial reservoir volume (ac-ft) | 5,170,000 |
| Final drawdown reservoir elevation (feet) | 1704 |
| Final drawdown reservoir volume (ac-ft) | 517,000 |
| Reservoir volume evacuated (ac-ft) | 4,653,000 |
| Drawdown period (days) | 123 |

# 6. SITE GEOLOGY AND GEOTECHNICAL DESIGN

## 6.1. General

Extensive site investigation, mapping, lineament studies and seismic hazard analysis etc. were carried out in the 1980s, and have been expanded and or revisited during the current initiative. However, the current projected site investigations will not be complete as originally planned and in time for the submission of the Draft Engineering Feasibility Report. Therefore, this section includes conservative geotechnical design parameters to be revised as necessary as interpretations are made of the geologic conditions, and characterization is performed.

## 6.2. Site Geology

A general description of the geology of the site and surroundings is given in Section 6 and in Appendix B1 of the Draft Engineering Feasibility Report.

## 6.3. Previous Exploration Programs

Geotechnical mapping and exploration of the dam site area for the Susitna-Watana Project have been carried out over a number of years beginning in 1975. A summary of these Project-specific geotechnical investigations are provided in the following reports.

Golder, 2013. Interim Geotechnical Data Report. Prepared for MWH Americas.

Fugro, 2013. Lineament Mapping and Analysis, TM-8. Prepared for MWH Americas.

Golder, 2012. Geotechnical Data Report, Watana Quarry A. Prepared for MWH Americas.

Harza-Ebasco, 1984. Susitna Hydroelectric Project, FERC Project No. 7114: 1984 Geotechnical Exploration Program, Watana Damsite. Volumes 1-2.

Harza-Ebasco, 1983. Susitna Hydroelectric Project: Watana Development Winter 1983 Geotechnical Exploration Program. Volumes 1-2.

Acres American, 1982. Susitna Hydroelectric Project: 1982 Supplement to the 1980-81 Geotechnical Report. Volumes 1-2.

Acres American, 1981. Susitna Hydroelectric Project: 1980-81 Geotechnical Report. Volumes 1-3.

USACE, 1979. South-central Railbelt Area, Alaska, Upper Susitna River Basin, Supplemental Feasibility Report, Hydroelectric Power and Related Purposes. Alaska District, Anchorage, Alaska.

USACE, 1975. South-central Railbelt Area, Alaska, Upper Susitna River Basin, Interim Feasibility Report, Hydroelectric Power and Related Purposes. Alaska District, Anchorage, Alaska.

## 6.4. Hydrogeologic Conditions

[To be completed.]

## 6.5. Geotechnical Parameters

### 6.5.1. Intact Rock Properties

#### *6.5.1.1. Andesite*

Index Properties

Unit Weight.....165 pcf
Specific Gravity .....2.7
Porosity .....1.8%

Intact Rock Strength and Deformation

Uniaxial Compressive Strength, σci .....10,000 psi
Elastic Modulus, Ei.....3.9 x $10^6$ psi
Poisson's Ratio, ν.....0.25
Friction Angle, φ..... $30^o$
Apparent Cohesion, ca .....TBD

#### *6.5.1.2. Diorite*

Index Properties

Unit Weight..... 165 pcf
Specific Gravity .....2.7
Porosity ..... 1.8%

Intact Rock Strength and Deformation

Uniaxial Compressive Strength, σci .....10,000 psi
Elastic Modulus, Ei..... 3.9 x $10^6$ psi
Poisson's Ratio, ν.....0.23

Friction Angle, φ ..... $30^o$
Apparent Cohesion, ca .....TBD

### 6.5.2. Rock Mass Strength and Deformation

#### *6.5.2.1. Rock Mass Shear Strength*

The shear strength of the rock mass will be calculated following the Hoek-Brown failure criterion (Hoek 2000). This procedure accounts for the shear strength of the intact rock as well as the shear strength of the discontinuities. The strength of the intact rock is represented by the unconfined compressive strength and by empirical values derived from triaxial testing of intact rock samples. The effect of the discontinuities is included by means of the geological strength index (GSI) which is derived from the RMR classification.

Controlled blasting methods will be specified for excavating sound bedrock; therefore, the zone of damaged rock mass is expected to be shallow, with some minor stress relief occurring within five to ten feet of the excavated surface and a Disturbance Factor D = 0.5–0.7 is expected.

#### *6.5.2.2. Modulus of Deformation*

Modulus of Elasticity will be determined by indirect and direct techniques. The indirect technique relies on the seismic velocities of the rock mass and rock mass rating 'GSI' values.

In the direct technique, the modulus of deformation will be determined from plate load tests carried out in the adits to be excavated at the dam site. Using the data obtained from this in-situ test, the modulus of elasticity will be obtained directly.

### 6.5.3. Rock Mass Poisson's Ratio

Poison's Ratio is a measure of the ratio of lateral strain over axial strain caused by loading of a rock mass usually varying from 0.20 for massive, high quality rock masses to 0.35 in more closely fractured, poor quality rock masses. A value of 0.25 is assumed apply to medium quality rock mass and 0.30 for poorer quality rock at the Susitna project.

### 6.5.4. Shear Strength of Rock Discontinuities

The Andesite and Diorite are intersected by several types of discontinuities (i.e., joints, bedding planes, foliation, etc.). For analyses involving translational sliding along discrete discontinuities, the orientation and characteristics of the specific discontinuity will be obtained from the work performed during site investigations.

The shear strength parameters of the discontinuities will be based on the Barton-Bandis non-linear strength criterion. The friction angle and apparent cohesion used in stability analyses are the "instantaneous" values taken as the tangent to the Barton-Bandis strength envelope for the applicable normal load associated with the particular loading condition being analyzed. The Barton-Bandis equation is given below:

$$\tau = \sigma_n \tan\left(\phi_r + JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right)\right)$$

Where:

$\tau$ = shear strength along the joint

$\varphi_r$ = residual friction angle

$JRC$ = joint roughness coefficient

$JCS$ = joint wall compressive strength

***Filled Discontinuities***

For joints and fault/shears with clay infilling and no rock-wall contact, the strength of the discontinuity will be governed by the infilling material.

Ice filled joints will be subject to special considerations. In general these features will only be considered unique in dilated joints where there is little rock to rock contact. In general ice will contribute very little shear strength and in any case ice will melt once joints around the dam are subjected to reservoir seepage. The shear strength of ice filled joints will be evaluated as a factor of the percentage of rock to rock contract, properties of other infillings and joint wall condition using the Barton-Bandis criterion discussed above. The primary impact of ice infillings is their (temporary) potential impact on rock mass permeability.

### 6.5.5. Rock Mass Permeability

The permeability of the rock mass is estimated from water pressure tests performed during the various site investigations and should be used to evaluate foundation seepage and tunnel inflows. The assumed permeability range is approximate. The range will be revised based on future water pressure testing.

Permeability, K $10^{-4}$ to $10^{-6}$ cm/s

Ice filled joints will be evaluated in the field in the exploratory adit and by permeability testing in boreholes.

### 6.5.6. Bedrock Concrete Interface

The shear strength of the concrete/rock foundation interface is a function of the roughness of the blasted foundation surface, the bond strength between concrete and rock and the compressive strengths of the RCC and bedrock. For the current Susitna designs, the shear strength of the concrete/rock interface will be assessed in accordance with precedent parameters used for large concrete gravity dams in similar geological conditions. Guidelines and precedent design values are presented in the 2004 ICOLD European Club publication *Working Group on Sliding Safety of Existing Gravity Dams, FINAL REPORT*, Coordinator: Giovanni Ruggeri (Italy).

Shear strength estimates will also use Barton's (1976)) methods for the computation of discontinuity shear strength when applicable. This technique utilizes laboratory testing, empirical data and field observations of rock and concrete strength parameters. Data will be gathered from additional field investigations including exploratory adits. It is useful for evaluating the nonlinear aspect of shear strength as well as checking values to use for design.

Interim values to be used for preliminary analyses are:

| | |
|---|---|
| Friction Angle, $\varphi_p$ | $45^o$ |
| Peak Cohesion, $c_p$ | 0-100 psi |
| Residual Friction Angle, $\varphi_r$ | $35^o$ |

### 6.5.7. Overburden

At time of writing no overburden has been tested. However, assumed material properties are summarized below and additional testing will be performed in subsequent phase of site investigations.

***Index Properties***

| | |
|---|---|
| Specific Gravity, Gs | 2.7 |
| Percent Passing No. 200 Sieve | 79 % |
| Liquid Limit, LL | 55 % |
| Plastic Limit, PL | 33 % |
| Plasticity Index, PI | 22 % |

| | |
|---|---|
| Average Natural Water Content, w. | 19.1 % |
| Unit Weight, γn | 120 pcf |

***Strength Parameters***

| | |
|---|---|
| Direct Shear Strength: | |
| Cohesion, c | 62 psf |
| Friction Angle, φ | 30° |
| Consolidated Drained Strength: | |
| Cohesion, c′ | 210 psf |
| Friction Angle, φ′ | 24° |
| Consolidated Undrained Strength: | |
| Cohesion, c′ | 170 pcf |
| Friction Angle, φ′ | 38° |

***Permeability***

| | |
|---|---|
| Overburden Soil Permeability | $6x10^{-6}$ cm/s |

***Consolidation***

Some portions of the overburden behave as normally consolidated and others as slightly overconsolidated. Parameters for both are provided although the selection of appropriate parameters should be based on conditions and subsurface profiles at the locations of interest. Refer to the boring logs in the GDR for detailed subsurface and stratigraphic information in specific areas.

| | |
|---|---|
| Preconsolidation Pressure | 16 psi |
| Compression Index, Cc | 0.2 |
| Recompression Index Cr | 0.0 |

***California Bearing Ratio***

| | |
|---|---|
| Select CBR at Top | 4.6 % |
| Select CBR at Bottom | 20 % |

***Dispersivity***

| | |
|---|---|
| Dispersivity | ND1 to ND3 |

### 6.5.8. Engineered Fills

Sources of engineered fill will be identified and evaluated from existing and ongoing site investigations. Engineered fill will be required as capping layers beneath structure foundations, beneath roads, backfill to retaining walls, etc. It should consist of durable well graded natural sand and natural gravel or crushed rock having a maximum particle size of 75mm.

All platform fill or filled embankments should either be retained within other fill materials or battered to a slope of compacted fill of no steeper than 1 Vertical (V) in 2 Horizontal (H) to prevent instability. All of the topsoil, organics and highly weathered soil must be removed, and the subgrade proof-rolled prior to any fill placement.

Typically an allowable bearing pressure of 3000 psf may be used for foundation design of structures founded on engineered fill. A minimum footing width of 20 inches is usually required.

# 7. SLOPE STABILITY

## 7.1. General

Slopes and excavations in rock and overburden materials will be classified as either “temporary” or “permanent”.

Temporary slopes and excavations are those that are required for construction but will be subsequently removed, backfilled or buttressed by concrete structures and fills. Temporary slopes that remain after completion of construction will meet the stability requirements for permanent slopes in the static, long-term, steady-state condition as described below.

Permanent slopes and excavations are those that will be exposed throughout the design life of the project and will form a permanent element of the work. Permanent slopes and excavations include, but are not limited to, the exposed excavations above the dam, powerhouse, portals, quarries and borrow areas, and access roads.

Design of the slopes will include appropriate application of material properties and strengths design loads, pore water pressures, and seismic loading. Temporary and permanent slopes and excavations will generally be designed and configured to meet the minimum slope stability requirements without assistance of active or passive supports. In cases where excavation support is necessary to achieve overall stability, the support type and capacity will be determined by stability analysis procedures defined below

## 7.2. Overburden / Fill Slope Stability Analysis

Stability analyses for slopes and excavations in soil and overburden materials will be performed by limit equilibrium methods using Spencer's method-of-slices to satisfy force and moment equilibrium. The analyses will be performed using the computer program SLOPE/W, a two-dimensional slope stability software product by Geo-Slope International (Calgary, Alberta).

The use of post-peak and "fully softened" strengths will be considered for excavations in clays and shales where slope movements or weathering of such materials may cause degradation of shear strength over time. For long-term slopes in clay, consideration will be given to the possibility that shear strength may degrade to residual values. Cracking at the top of slopes in clayey material will also be considered.

For slopes on soft soils or loose silts the liquefaction potential under seismic conditions will be evaluated using the SPT-based method recommended by Idriss and Boulanger (2008) and Seed (2010) for granular soils, and the index property based method recommended by Bray and Sancio (2006) for fine grained soils.

## 7.3. Rock Slope Stability Analysis and Support

Excavations in bedrock will be carried out by mechanical and /or blasting techniques. Final, permanent slopes will be constructed by controlled blasting techniques.

Stability analyses in rock will be performed for bench-scale (local) stability as well as overall (global) slope stability. Strength parameters for rock masses and discontinuities used in stability analyses shall be established as per the procedures outlined in Sections 6 above.

The stability of large and small surface rock excavations at Susitna Watana is controlled by sliding and/or toppling of structurally controlled wedges and blocks that are released by the creation of the excavation. Localized rock mass overstressing may play some role in deep seated instability of very high slopes. The design of appropriate slope angles and rock support will include the following tasks.

- Kinematic analyses will be carried out using Rocscience DIPS software. These analyses will use joint survey data and will identify any structurally defined slope stability mechanisms that may be present. Failure modes would consist of one or more planar blocks defined by a single set of discontinuities, wedge sliding along two intersecting sets of discontinuities and toppling along steeply inclined discontinuities.
- The analyses of wedge sliding mechanisms will be carried out with Rocscience SWEDGE software.

- Planar siding analyses will be carried out with hand calculations and by Rocscience ROCPLANE software.
- Rotational sliding analyses will consider rock mass strength parameters and be performed utilizing Spencer's method-of-slices or similar methods that satisfy force and moment equilibrium. The analyses, for either circular or non-circular sliding planes, will be performed using the computer program Slide by Rocscience or SLOPE/W, by Geo-Slope International (Calgary, Alberta) as appropriate.

In areas of local or large scale instability support measures such as soil/rock anchors and shotcrete will be installed to establish an appropriate factor of safety against failure, maintain the integrity of the excavation and prevent progressive deterioration. Protection measures such as wire meshing and parapet walls, will be utilized as necessary to provide long-term protection for rockfalls and other falling debris.

Rock bolts and high-capacity tendons will be designed in accordance with the Post-Tensioning Manual (PTI 2006).

Rock bolts for support of permanently exposed surface slopes in bedrock will be 38 mm diameter or less, galvanised steel threaded bars with a yield strength not greater than 517 MPa. All rock bolts will be fully grouted or epoxy encapsulated for corrosion protection. Rock bolts will have a design working load not exceeding the lesser of one-half (½) of the ultimate strength of the rock bolt or two-thirds (2/3) of the yield strength of the rock bolt. Where rock bolts are required to prevent toppling or wedge failures, a minimum factor of safety of 2.0 will be used to determine the number, location and orientation of rock bolts.

Permanent excavations will be drained and protected from seepage/infiltration, as necessary, with inclined drain holes installed in the slope face and lined gutters at the work/access benches. Peripheral drainage ditches will be provided at the top of the excavations to control surface water and minimize infiltration.

## 7.4. Seismic Loads for Stability Analysis

For intermediate and final design phases, seismic analysis of temporary and permanent slopes will be performed using pseudo-static methods, simplified deformation analyses in accordance with Makdisi and Seed (1977) or Newmark (1965), or detailed dynamic analysis utilizing the finite-difference numerical modeling software such as FLAC, by Itasca (Minneapolis, MN). OBE seismic peak ground acceleration (PGA) and pseudostatic, as detailed in Table 10-1, shall be used in the analyses as applicable. The selection of the appropriate method of analysis will be based upon the performance requirements for the slopes or excavations under consideration.

## 7.5. Numerical Modeling

Where numerical modeling is necessary to evaluate slope stability, stresses and deformations, progressive failure mechanisms or ground-structure interactions, the analyses will be carried out using two-dimensional or three-dimensional elasto-plastic finite element and finite difference methods. The software products employed for numerical modeling will include ABAQUS (Simulia), FLAC (Itasca) and Phase 2 (RocScience).

## 7.6. Phreatic Surfaces for Stability Analyses

Groundwater levels and phreatic surfaces used in the stability analyses of overburden and rock slopes and underground excavations shall be based on piezometric measurements in boreholes and seepage analyses taking into account any drainage to be provided.

## 7.7. Slopes for Excavations and Fills

Slopes in excavations and placed fills will be in accordance with the guidelines presented in the following sub sections.

### 7.7.1. Bedrock Excavations

Excavated bedrock slopes, excluding the pre-excavated plunge pool, will be cut to the following maximum inclinations:

- Widely jointed igneous and metamorphic bedrock: 0.15h:1.0v to 0.25h:1.0v
- Highly fractured and weathered bedrock: 0.50h:1.0v to 1.0h:1.0v
- Slopes, against which concrete structures will be constructed, may be vertical for the height of such structures.

Bedrock slopes in any pre-excavation associated with the plunge pool will be sloped 0.5H:1.0v to 1.0H:1.0V, as warranted by the structural geology and the competence of the rock mass.

All overburden and loose debris shall be cleared from the rock surface for a minimum width of 10 ft. from the top of slopes excavated in rock. Appropriate measures shall be installed within the cleared zone and/or over rock faces to permanently control rock falls, ice falls, snow and surface water run-off from impacting structures or operations below the top of the slope.

### 7.7.2. Overburden Excavations

In overburden the following slope angles will be used for excavation designs:

a) Groundwater table beneath the base of the excavation:

- Granular soils (mixtures of sand gravel and cobbles): 1.5h:1.0v
- Silty/clayey sand and/or gravel: 2.0h:1.0v
- Sand: 2.0h:1.0v

b) Groundwater table above the base of the excavation:

- Granular soils (mixtures of sand gravel and cobbles): 3.0h:1.0v
- Silty/clayey sand and/or gravel: 4.0h:1.0v
- Uniform Sand: 4.0h:1.0v

### 7.7.3. Compacted Fills

Compacted fills will have the following slope gradients.

**Table 7-1. Recommended Slopes for Different Soils**

| Fill Type | Slope Gradient (H:V) | |
|---|---|---|
| | Slope not subject to inundation | Slope subject to inundation |
| Hard, angular rockfill, blasted or ripped | 1.5:1.0 | 1.5:1.0 |
| Well graded gravel | 1.5:1.0 | 2.0:1 .0 |
| Poorly graded gravel, sand | 1.5:1.0 | 2.0:1.0 |
| Silty/clayey gravel | 1.8:1.0 | 3.0:1.0 |

## 7.8. Minimum Factors of Safety for Excavations and Fills

The following minimum factors of safety will be provided for permanent excavation and fill slopes in soil and rock with the combination of reservoir levels or piezometric levels that yield the worst case scenario:

| | |
|---|---|
| Under steady state seepage with drainage and long-term conditions | 1.5 |
| During construction | 1.3 |
| During the OBE Earthquake | 1.1 |

## 7.9. Slope Protection

Temporary and permanent slopes in materials that may be subject to erosion from surface runoff, or deterioration from atmospheric exposure, will be protected with vegetation cover, stone facing, or shotcrete, as appropriate.

Slopes subjected to the action of fast moving water will be protected by riprap with appropriate bedding and/or grouted riprap. Riprap will be designed and sized to resist wave action and erosion for the applicable loading conditions.

Riprap and erosion protection will be designed in accordance USBR Design Standards No. 13, Embankment Dams, Chapter 7 - Riprap Slope Protection or USACE Engineer Manual EM-1110-2-1614 Design of Coastal Revetments, Seawalls and Bulkheads.

## 7.10. Seepage Analysis

Seepage analyses necessary to support stability analyses and design drainage and dewatering systems will be performed by numerical modeling. The software product employed for numerical seepage analyses will be SEEP/W (Geo-Slope International). Where slope or tunnel stability is being evaluated using other programs, coupled seepage-stability analyses will be performed.

Seepage analyses will be validated with respect to relevant groundwater data and pumping test data where available.

# 8. FOUNDATIONS – DAMS AND STRUCTURES

## 8.1. Bearing Capacity of Foundations on Rock

The bearing capacity of rock foundations will be determined in accordance with EM 1110-1-2908 - Rock Foundations or the approach described in Wyllie (1992) based on anticipated rock mass strength and conditions, applicable loading conditions, and likely failure modes.

For eccentric loading conditions, the ultimate bearing capacity of the foundation will be determined by distributing the vertical force component uniformly across a reduced effective foundation width defined by the following equation:

$$B' = B - 2e$$

Where:

$B'$ = effective base width

$B$ = foundation base width

$e$ = eccentricity

The effective base width ($B'$) is used in the appropriate bearing capacity equation to compute the ultimate bearing capacity.

The allowable bearing capacity value ($q_a$), based on the strength of the rock mass, is defined as the ultimate bearing capacity, $q_{ult}$, divided by the factor of safety, $FS$.

$$q_a = q_{ult} / FS$$

The minimum allowable bearing capacity for foundations will incorporate a factor of safety of 2.5. The average bearing pressure acting on the foundation material must be equal to or less than the allowable bearing capacity. Foundation bearing pressures will be computed using Boussinesq's formula, analytical methods, or numerical methods.

## 8.2. Bearing Capacity of Foundations on Soils

The ultimate bearing capacity of soils and fills will be determined in accordance with EM 1110-1-1905 Bearing Capacity of Soils, with consideration to the applicable loading conditions, geologic conditions, size/shape and depth of foundations.

The minimum allowable bearing capacity for foundations will include a factor of safety of 2.5. The average stress acting on the foundation material must be equal to or less than the allowable bearing capacity.

## 8.3. Lateral Earth Pressures

For static and pseudo-static analyses, lateral pressures on structures and walls will be computed using Rankine or Coulomb and Mononobe-Okabe methods, respectively, and the following coefficients:

$\phi_d = 2/3\ \phi'$

$K_a = \tan 2\ (45° - \phi_d/2)$

$K_p = \tan 2\ (45° + \phi_d/2)$

$K_o = 1 - \sin \phi$

Earthquake effect will be calculated using the Mononobe-Okabe procedure. Hydrodynamic effects will also be included.

## 8.4. Settlement

Settlement of foundations and fills will be determined in accordance with EM 1110-1-1904 Settlement Analysis, with consideration to the applicable loading conditions, geologic conditions, size/shape and depth of foundation. Settlement computations will utilize computer spreadsheets and/or the 2-dimensional numerical modeling such as Phase2 by RocScience (Toronto, Ontario), 3-dimensional computer program such as Settle3Dfor the analysis of initial (elastic) settlement, consolidation, and time-dependent settlement under foundations, embankments and surface excavations. The stress distribution due to loading will be computed using the Boussinesq or other applicable methods.

## 8.5. Dam Foundation Preparation

The excavation for the dam foundation will be carried out down to a level where the rock is competent enough to support the loads of the dam. Geological site investigation data, including boreholes, geophysics and adit mapping will be used to determine the depth of foundation excavations. Control blasting will be specified for the excavation and foundation treatment would include dental concrete, consolidation and curtain grouting.

The requirements for shaping of the RCC dam foundation will be based on considerations of stresses within the dam. Based on finite element analyses of the dam, broad criteria shall be defined for optimum foundation shape and orientation.

Overhangs and high vertical faces will not be permitted. In this regard, the steep, almost vertical walls of the lower river valley should be excavated to a suitable inclination to minimize stress concentrations in the dam body. This shall be further refined through finite element analyses during detailed design.

The requirements for dental excavation will be based in part on the likelihood of stress concentrations developing in the dam due to the presence of soft zones and also on considerations of seepage and piping of soft or weak material in the foundation. The required depths of dental excavation and dental backfill concrete will be based on considerations of stress transfer and arching of concrete within concrete to zones of higher modulus rock on either side as discussed in USBR (1977). Structure specific criteria relating depth of excavation to width of the feature shall follow guidelines presented by USBR (1977).

## 8.6. Dam Foundation Grouting

Grout injections will use single "stable" mixes and closure criteria will follow the GIN procedure (Lombardi and Deere 1993) or other recognized criteria (ASCE). Both injection pressure and volumes of grout will be controlled according to recognized procedures. Water pressure testing will be carried out in check holes to confirm that after grouting rock permeability is of the order of 2 to 3 Lugeon value.

Consolidation grouting will be carried out in the dam foundation across the entire dam footprint to remediate any damage caused by blasting and stress relief during foundation excavation and to provide an extra degree of water seepage control in the foundation rock at the base of the dam. Grout holes will be drilled on a regular grid across the entire bearing surface of the dam foundation. Grout hole depth will be determined by geological conditions and dam foundation requirements in accordance with precedent practice.

A grout curtain will be provided in the foundation rock to control seepage through the foundation bedrock and reduce uplift pressures at the base of the dam. The grout curtain will consist of a single or multiple rows of vertical or inclined, split-spaced primary holes and corresponding secondary, tertiary and other stages till closure criteria is met.

Grouting layout designs for the Susitna-Watana project will be in accordance with precedent experiences in similar dam schemes with compatible geological conditions. Grouting will be carried out through holes drilled from the dam foundation and from galleries within the dam body and in the rock abutments.

Based on mapping and investigation data, grout holes shall be oriented to intersect the maximum number of discontinuities. The final orientation of the grout curtain under the dam shall also consider the effect of the orientation of the abutment and sliding stability of the structure and rock mass under the foundation.

The spacing of primary grout holes shall be selected such that adequate coverage of every section of the grout curtain is covered by drilling and grouting, even if there are no significant takes in a given section. It should be specified that that selected primary grout holes shall be drilled in excess of the specified grout curtain depth to ensure that there are no unexpected pervious features present in these areas. These selected or "exploratory grout holes" shall not be spaced less than 75 ft. apart.

The criteria for acceptance testing of the grout curtain shall be based on cored boreholes drilled within the grout curtain to sample grouted rock. The core shall be logged in detail, including a

complete discontinuity log. Water pressure testing shall be carried out in the exploratory boreholes and the Lugeon value should not exceed the following limits:

- 95% of the tests shall not exceed 3 Lugeons
- 100% of the tests shall not exceed 8 Lugeons.

Special grouting measures will be devised to counteract ice filled joints in the zone of stress relief if deemed necessary.

## 8.7. Dam Foundation Drainage

Drainage galleries and drainage curtains consisting of vertical or inclined drainage holes will be provided. These will generally be drilled into the foundation from drainage galleries in the dam body and in the rock abutments, to intercept seepage and reduce uplift pressure. The spacing and depth of the foundation drainage holes would be decided based on the characteristics of the rock mass, particularly its permeability and discontinuity pattern. Water collected in the galleries will be conveyed downstream by means of discharge galleries.

Drain holes will only be drilled after completion of grouting in each area of the foundation.

Drain holes will be provided with collars and standpipes to allow measurement of water pressure by connecting a pressure gauge to the tops of selected holes.

## 8.8. Dam Foundation and Abutments Sliding Stability

Foundation sliding stability of the gravity dam will be analyzed. Three separate sliding stability cases will be considered:

- Cross valley stability of dam blocks during construction. This applies to temporary conditions where blocks have been partially constructed on the sloping abutments prior to placement of central blocks which can restrain downslope sliding. In general, this sliding analysis will focus on the impact rock mass shear strength and of unfavorably oriented discontinuities in the abutments.
- Two-dimensional sliding of the dam towards the downstream direction when acted upon by reservoir loadings, uplift and earthquake loadings as appropriate for the analytical case. Two potential sliding planes will be considered (a) concrete/rock interface at the base of the dam and (b) sliding of two dimensional wedges or blocks formed by sub horizontal and inclined discontinuities in the rock beneath the foundation of the dam.

- Three-dimensional wedge sliding of tetrahedral blocks in the abutments. The abutment foundations are particularly critical to the stability of a curved RCC dam where arch type thrust loadings can develop in the abutments. Abutment instability can develop along either a planar discontinuity or a combination of planar discontinuities which intersect to form an unstable wedge.

All analyses of foundation rock blocks or wedges will consider that the sliding rock body is a rigid mass and defined by one or more continuous joint plane(s). Two-dimensional sliding analyses will be carried out by standard shear friction methods as detailed by UASCE in EM 1110-2-2200 "Gravity Dam Design". Simpler configurations will be analyzed with CADAM 2004 or similar software. Sliding along deeper seated subhorizontal discontinuities in the rock mass beneath the dam can be analyzed by CADAM and Rocscience SLIDE software.

Large, three-dimensional tetrahedral wedges may be identified in the abutments by assessment of structural geology information. Kinematic assessments will be carried out to determine if any of these features are susceptible to sliding when acted upon by loadings from the dam and piezometric uplift. Londe type three-dimensional limit equilibrium analyses will be carried out to analyze the sliding stability of any features that are consider to be kinematically susceptible to sliding. Analyses will be carried out with RIGID software, supplemented by graphical type stereonet vector analyses as per the methods of Londe et al (1069) and Goodman (1989). The limit equilibrium analysis will be supplemented by three-dimensional finite element models, if warranted by the initial limit equilibrium results.

Design factor of safety values are as follows:

- The analyses of cross valley sliding of dam blocks are for a temporary condition and these features must have a normal loading condition factor of safety equal to 1.3 or greater.
- Seismic displacements using Newmark type sliding block analyses or similar methods must conform to acceptable limits.

## 8.9. Plunge Pool Scour

Scour will occur at the plunge pool. Following the initial decision on spillway orientation and flip bucket throw, detailed assessments will be made of plunge pool scour depth primarily to confirm if this excavation can be unlined. The study will review the erodibility of the rock mass and compute scour depths using two or more internationally accepted sour depth formulas.

Scour of a rock mass occurs largely by plucking of joint bounded rock blocks from the rock mass. Joint spacing, block size and the amount of rock mass disturbance are important factors in

determining the scour potential of a rock mass. The first step of the erodibility analysis will be to produce a rock mass model of the rock affected by spillway flows. This will include a compilation of geology, rock mechanics testing data and structural features such as joints faults and major discontinuities. Rock mass assessment will use information from the field mapping and subsurface investigations and will include GSI (RMR89) and Q rock mass classifications. These classifications give empirical estimates of rock mass strength using use rock properties that are critical for erosion resistance.

The second step is to compute maximum scour depth using accepted scour erosion formulae.

The feasibility study scour assessments will include the following:

- **Annandale Erodibility Index (1995):** The Annandale is the best documented method that uses rock properties. This method assumes that there is a relationship between the rate at which energy is dissipated in the receiving pool of water (the stream power P) and the erodibility of the rock. The Erodibility Index, which is based on a rippability index original developed by Kirsten (1982), is as follows:

  $$Kh = MsKbKdJs$$

  Where Kh is the erodibility index, Ms is the mass strength number (based on compressive strength of the rock), Kb is the particle block, size number, Kd is the discontinuity or inter-particle bond shear strength-number and Js is the, relative ground-structure number that is based on the orientation of the primary discontinuities'. Higher values of the Erodibility Index indicate greater resistance to erosion. The parameters are closely based on the Q evaluation and these are easily converted. The Kb erodibility Index is calculated as follows:

  $$Kb = RQD/Jn$$

  Where Jn is based on the number of joint sets, Kd is evaluated in terms of the ratio. Jr/Ja where Jr is the joint roughness number and Ja is joint alteration number (similar to Q system). The various parameters are evaluated by means of simple tests and the set of tables published by Annandale for each parameter.

  Together with an estimate of plunge pool stream power and energy dissipation, the erodibility index is used to evaluate if erosion of the plunge pool is likely. Figure 8-1 shows the relationship between erosion, erodibility index and energy dissipation.

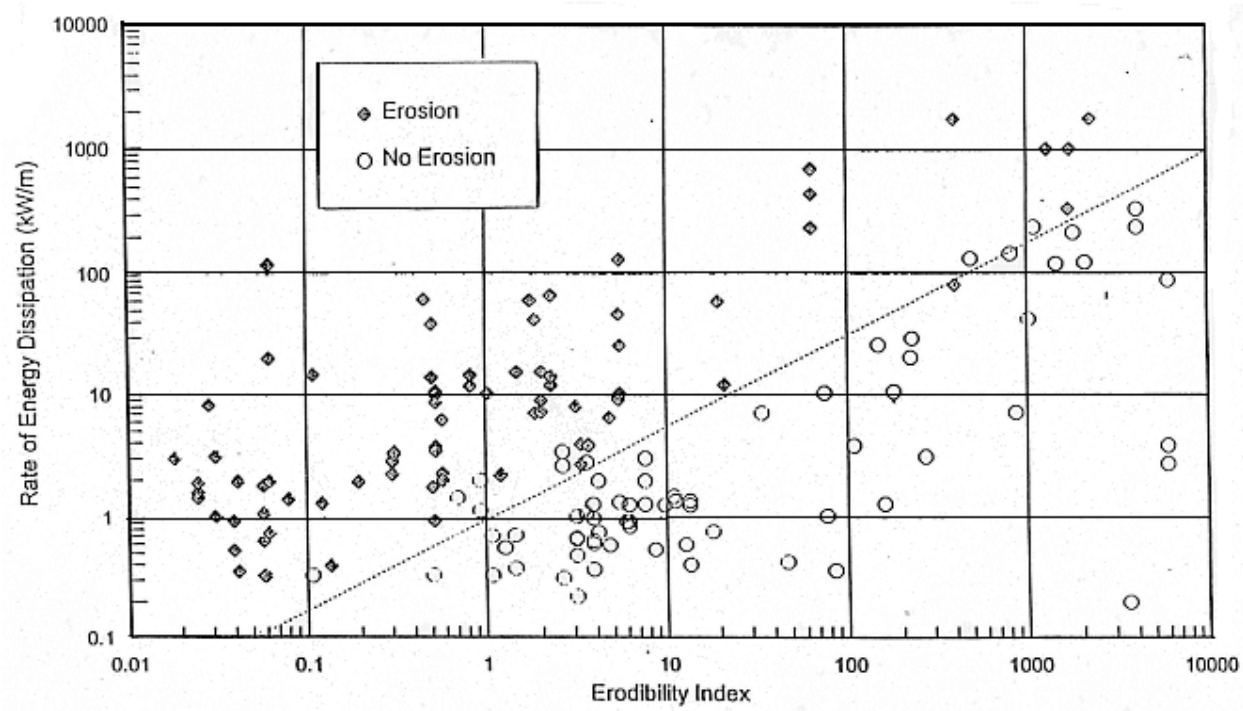


**Figure 8-1. Erodibility of Earth and Rock Materials by Hydraulic Jets (Annandale 1995)**

- **Maximum Scour Depth:** A number of maximum scour depth relationships are in use. All are empirical and most are based on assessments of actual case histories. The 1994 Yildiz version of 1937 Veronse formula will be used.

$$D = 1.32\ H^{0.225}\ q^{0.54}\ \text{Sin}\ \Theta$$

Where D is the maximum scour depth (FT), H is the difference in elevation between the reservoir and the tailwater surfaces (FT), q is the unit discharge (CFS) and Θ is the angle of inclination of the jet at the water surface in the plunge-pool.

A second formula, the Mason formula, will also be employed. This formula relates depth of scour to unit flow, head drop, acceleration of gravity, tail-water depth and particle size of bed material. It relates scour depth to the theoretical particle size of the plunge pool bed material. It has been calibrated by model studies and actual plunge pool performance. This relationship is:

$$D = 3.72\ \frac{q^{0.60} H^{0.05} h_2^{.015}}{g^{0.30} d^{0.10}}$$

Where q is the unit discharge, g is the acceleration due to gravity, d is the mean rock size, and h2 is the tailwater depth.

## 8.10. Lateral Earth Pressures

For static and pseudo-static analyses, lateral pressures on structures and walls will be computed using Rankine or Coulomb and Mononobe-Okabe methods, respectively, and the following coefficients:

$\phi_d = 2/3\ \phi'$

$K_a = \tan 2\ (45° - \phi_d/2)$

$K_p = \tan 2\ (45° + \phi_d/2)$

$K_o = 1 - \sin \phi$

Earthquake effect will be calculated using the Mononobe-Okabe procedure. Hydrodynamic effects will also be included.

## 8.11. Settlement

Settlement of foundations and fills will be determined in accordance with EM 1110-1-1904 Settlement Analysis, with consideration to the applicable loading conditions, geologic conditions, size/shape and depth of foundation. Settlement computations will utilize computer spreadsheets and/or the 2-dimensional numerical modeling such as Phase2 by RocScience (Toronto, Ontario), 3-dimensional computer program such as Settle3Dfor the analysis of initial (elastic) settlement, consolidation, and time-dependent settlement under foundations, embankments and surface excavations. The stress distribution due to loading will be computed using the Boussinesq or other applicable methods.

# 9. UNDERGROUND EXCAVATIONS

The underground excavations required for the Susitna-Watana project include one diversion tunnel; drainage tunnels on either abutment; and access tunnels. Each underground excavation will be designed considering the following criteria.

## 9.1. Rock Support Design

The tunnels and shafts will be constructed in Andesite and Diorite. Fair to good rock conditions are expected within most of the tunnel and shaft excavations. Rock bolts and shotcrete will provide adequate primary rock support along the majority of the tunnel alignment; however, fault

and shear zones may require additional rock support such as ribs, concrete linings, or other remedial measures. The underground excavations are at relatively shallow depths such that overstressing related issues are expected to be minimal, although low confining stresses at shallower sections (i.e., near portals) may necessitate extra rock support locally.

Rock mechanics design work for the underground excavations will include the following analytical work items:

- Delineate geotechnical zones of rock mass conditions and geological features along the tunnel routes.
- Assess in situ stresses together with rock mass strength, deformability, and Poisson's ratio,
- Estimates of rock support will be made using empirical methods such as Barton's Q rock support classification system. Bieniawski's RMR classification system will be used to provide supplementary rock mass classification and rock support estimates.
- Rock mass strength estimates will be made using Hoek's GSI rock mass classification system. The results of in situ testing and geologic mapping will be incorporated into this assessment. Interpretations made on the properties of the in situ rock and rock mass will account for the potential change in these properties during excavation, caused by loosening of the rock mass, blast damage, stress changes, groundwater changes, etc.
- Two-dimensional finite element analyses will be performed, if necessary, of selected tunnel sections using the Phase2 finite element modeling program (Rocscience, Inc.) to evaluate the stability of the rock pillar between parallel tunnels and at tunnel intersections. These analyses will appropriate non-linear strength envelopes and rock mass deformability parameters as determined by field assessment of the applicable ground profile, rock masses, laboratory testing, Hoek's GSI classification and analyses using RocLab software (Rocscience, Inc.).
- The Phase2 analyses, together with RocSupport (Rocscience, Inc.) limit equilibrium analyses, will estimate the expected deformation behavior of the various support systems. These target deformations will be compared to actual deformations during tunnel construction as part of the assessment of the performance of rock support systems and tunnel stability.
- Wedge stability analyses for the tunnels and shafts areas will be carried out with the UNWEDGE program (Rocscience, Inc.). If deemed necessary by this analysis, rock

support designs will be modified in appropriate areas to stabilize potential blocks and wedges that do not meet appropriate factors of safety.

- Miscellaneous limit equilibrium analyses will be carried out as required.

Geotechnical design will be in accordance with accepted rock/soil mechanics practices and the criteria outlined in the ASCE document *Civil Engineering Guidelines for Planning and Designing Hydroelectric Developments, Volume 2, Waterways* (1989).

Rock support designs will be developed for various categories of ground conditions. Rock support design will consider the behavior of the overall rock mass as well as special requirements to support structural geology features such as wedges and sliding blocks.

Rock support will be mainly provided by rock bolts and shotcrete. Ungrouted mechanical bolts, "split set" bolts and "Swellex" bolts will be permitted for use only as temporary rock support applications. The design of the excavation and support system of the underground excavations will allow the excavated faces to deform in a controlled manner to the extent that practically feasible support pressure intensities from the support system can be applied during and after excavation. These must be replaced by permanent fully grouted rock bolts to provide permanent support as required.

Permanent rock support will be designed to provide long term stability. The rock support systems will take up all permanent rock loads. Consequently, concrete and steel linings that are installed for hydraulic purposes or to other embedded structures in the tunnels and shafts will not be subjected to rock loads.

## 9.2. Tunnel Dewatering Estimates and Design

Tunnel dewatering requirements will be estimated. Overall rock mass permeability is expected to be low but a number of zones of highly fractured or faulted rock should be expected to discharge groundwater flows into the underground excavations.

Permeability data from completed and future investigations and permeability testing, piezometric levels and subsurface geology information will be used to estimate seepage amounts. The criteria of Heuer ("Estimating Rock Tunnel Inflow 1995, Volume 1 from 1995 RETC and Volume II from 2005 RETC) will be used to provide overall estimates. The Heuer criteria uses drillhole permeability data and estimates water inflows from distributed, interconnected discontinuities. The permeability is input as a histogram distribution of permeability values in order to evaluate the effects of permeability variability (average equivalent permeability is not used since this masks the significance of rock mass variability).

Two values of inflow will be computed:

- Heading "flush" inflow. This is a short term initial inflow that occurs immediately after a permeable, water bearing zone is intersected. Flush flow bleeds off with time; significant reductions occurring over a period of days, sometimes hours. This parameter enables estimates to be made for short term water handling facilities at the tunnel face.

- Long-term steady-state flow: Steady-state flow develops some time and distance behind the heading. It refers to conditions that exist during the time of construction, a period of months to years.

Tunnel drainage will be affected by a combination of sump pumps in the underground excavations and gravity drainage through previously excavated access tunnels and adits.

# 10. SEISMICITY

## 10.1. Seismicity Studies

Seismic hazard investigations have been performed for the Susitna Watana project. The primary objective of these studies is to establish the seismic design criteria for the project and to determine expected ground motion parameters for use in design of the dam and related structures.

The seismic studies are described in Appendices B2 and B3 of the Draft Engineering Feasibility Report and selection of seismic design criteria is discussed in Section 10 of the main report.

## 10.2. Seismic Criteria

Using data from the seismic hazard investigations, the following earthquake cases defined by the USACE will be finalized. These are described as follows:

a) **Maximum Credible Earthquake (MCE):** This earthquake is defined as the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence. Since a project site may be affected by earthquakes generated by various sources, each with its own fault mechanism, maximum earthquake magnitude, and distance from the site, multiple MCE's may be defined for the site, each with characteristic ground motion parameters and spectral shape. The MCE is determined by a Deterministic Seismic Hazard Analysis.

b) **Maximum Design Earthquake (MDE):** The MDE is the maximum level of ground motion for which a structure is designed or evaluated. The associated performance requirement is that the project performs without significant failure, such as uncontrolled release of a reservoir, although severe damage or economic loss may be tolerated. For critical features such as the dam structure, the MDE is the same as the MCE. For all other features, the MDE shall be selected as a lesser earthquake than the MCE which provides economical designs meeting appropriate safety standards. The MDE can be characterized as a deterministic or probabilistic event.

c) **Operating Basis Earthquake (OBE):** The OBE is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50-percent probability of exceedence during the service life (This corresponds to a return period of 144 years for a project with a service life of 100 years.). The associated performance requirement is that the project functions with little or no damage, and without interruption of function. The purpose of the OBE is to protect against economic losses from damage or loss of service, and therefore alternative choices of return period for the OBE may be based on economic considerations. The OBE is determined by a probabilistic seismic hazard analysis (PSHA).

The following parameters will be developed for each design case and submitted to project designers:

- Seismic source characterization and earthquake magnitude. Faults that may affect the damsite will be identified.
- Ground motion parameters, including Peak Ground Acceleration (PGA) values.
- Uniform hazard spectra (UHS) from a PSHA or response spectrum from a deterministic analysis, where appropriate.
- Contribution by source, magnitude and distance ranges, and magnitude-distance de-aggregation for the PSHA analyses.
- Acceleration time histories. A minimum of three time histories will be provided for each MCE earthquake case. Time histories will be spectrally matched, from recorded time histories of earthquakes that are from the same type of event (interface, intraslab or crustal), and have comparable magnitude and distance to the earthquake event.

Preliminary earthquake design parameters used for the Susitna-Watana project are listed in Table 10-1. These will be subject to revision once the seismic hazard evaluation has been completed, and following discussion with FERC. FERC will be expected to endorse the choice of parameters. Design spectra and time history graphs and data tables will be produced during the finalization of the seismic hazard studies.

**Table 10-1. Preliminary Seismic Design Parameters for Susitna Structures and Excavations**

| Earthquake Case (and associated return interval) | Type of Earthquake | Earthquake Magnitude ($M_w$) | Horizontal Peak Ground Acceleration PGA, (g) | Pseudostatic Coefficient, k (g) |
|---|---|---|---|---|
| Maximum Credible Earthquake (MCE) | Intraslab | 8.0/7.5* | 0.81/0.69 | n.a. |
| | Interface | 9.2 | 0.58 | n.a. |
| | Crustal | 7.0 | 0.49 | n.a. |
| Maximum Design Earthquake (MDE), 1/2500 years | - | - | 0.52 | n.a. |
| Operating Basis Earthquake (OBE)<br>• 1/1000 years (structures and some critical slopes)<br>• 1/500 years (most slopes) | <br><br>-<br>- | <br><br>-<br>- | <br><br>0.37<br>0.27 | <br><br>0.19<br>0.14 |

*Intraslab includes both a M7.5 and M8.0.

Horizontal and vertical response spectra along with time histories will be provided for two dimensional and three dimensional stability analyses.

MDE and OBE parameters will be used for seismic stability analyses of the dam and all related water retaining structures (gate guides, spillways, etc.). The stability of the dam structure and gates will be analyzed by dynamic methods and appropriate spectrally matched time histories.

OBE parameters will be used for all ancillary structures whose failure would not compromise the ability of the project to retain the reservoir. In addition to the horizontal response spectrum a vertical response spectrum will be developed to aid in the design. Facilities that will be analyzed for the OBE event will include:

- Natural and excavated slopes of the power facilities.
- Diversion works areas away from the dam.
- All structures and components related to power production (penstock, power intake, powerhouse, tunnels, caverns, turbines, switchyard, transmission lines, water supply).

The OBE analyses of critical mechanical and civil structures will use the 1/1000 year peak ground motion parameters and response spectra. Natural and excavated slopes at the damsite

and in the reservoir will be analyzed by pseudostatic methods using the OBE pseudostatic coefficient for the 1/500 and/or the 1/1000 year event as deemed necessary. If necessary, the deformations of slopes shall be evaluated with PGA parameters using the following pseudo-dynamic methods:

- Makdisi and Seed (1978) method
- Jansen (1990) method

# 11. GENERAL STRUCTURAL CRITERIA

## 11.1. Structural Analysis and Design

Structural analysis and design will be carried out in accordance with U.S., Alaska and internationally recognized standards and relevant technical publications. The analysis will determine the critical load case for each structure and structural member, verify the stability of each structure and calculate the critical loads to be carried by each of the structural members. The design will size each structure and structural member to safely and economically carry the required load using the prescribed materials.

## 11.2. Material Properties for Design

The material properties selected for conceptual design are listed below and will be further refined following field investigations and laboratory testing.

### 11.2.1. Water

| | |
|---|---|
| Unit weight | 62.4 pounds per cubic foot (pcf) |
| Quality | (Untreated water for concrete production) |

### 11.2.2. Structural Concrete

| | |
|---|---|
| Unit weight | 150 pcf |
| Unconfined compressive strength | f'c = 4,000 psi |
| Coefficient of thermal expansion | 0.0000055/degrees F |

### 11.2.3. Steel Reinforcement

ASTM A615, Grade 60

## 11.3. Design Loads

The structures will be designed for the following loads where applicable. The loads will be combined as described in Section 11.4 and as defined in applicable reference standards. Loads and load cases for the RCC dam are further defined in Sections 13.3 and 13.4.

- Dead loads
- Live loads
- Hydrostatic pressure
- Hydrodynamic pressure
- Uplift pressure
- Transient pressure
- Equipment loads
- Earth pressure
- Wind loads
- Seismic loads
- Snow loads
- Ice pressure
- Temperature loads
- Equipment impact loads
- Impact loads from flood debris.

### 11.3.1. Dead Loads

Dead loads to be considered in the design will include the weight of the structure, including walls, floors, partitions, roofs, and all other permanent construction and fixed equipment. The following unit weights of materials will be used:

| Material | Weight | Unit |
|---|---|---|
| Mass concrete | 150 | lbs./ft$^3$ |
| Reinforced concrete | 150 | lbs./ft$^3$ |
| Steel | 490 | lbs./ft$^3$ |
| Water | 62.4 | lbs./ft$^3$ |

| | | | |
|---|---|---|---|
| Silt | - vertical | 120 | lbs./ft$^3$ |
| | - horizontal | 85 | lbs./ft$^3$ |
| Backfill (to be verified by geotechnical data) | | | |
| | - dry | 115 | lbs./ft$^3$ |
| | - saturated | 130 | lbs./ft$^3$ |
| | - submerged | 70 | lbs./ft$^3$ |

### 11.3.2. Live Loads

The following live loads will be used for design of slabs, beams, columns and other structural members in the areas indicated. Concentrated loads will be added in designated areas as appropriate.

**Table 11-1. Live Loads**

| Area | Live Load |
|---|---|
| Roofs | 20 lbs/ft$^2$ |
| Stairs and egresses | 100 lbs/ft$^2$ |
| Handrails, guardrails, grab bars | Per ASCE 7-10 Section 4.4 |
| Offices, corridors, locker room floors | 100 lbs/ft$^2$ |
| Equipment and Storage room floors | 200 lbs/ft$^2$ |
| Control room floor | 200 lbs/ft$^2$ |
| Generator room floor | 1000 lbs/ft$^2$ |
| Turbine room floor | 500 lbs/ft$^2$ |
| Service/erection bay floor | 1500 lbs/ft$^2$ |
| Transformer areas | 300 lbs/ft$^2$ |
| Intake deck - general | 300 lbs/ft$^2$ or AASHTO HS-20 |
| Intake deck – heavy lift areas | 1000 lbs/ft$^2$ |
| Gantry deck | 300 lbs/ft$^2$ or AASHTO HS-20 |
| Powerhouse access | 300 lbs/ft$^2$ or AASHTO HS-20 |
| Draft tube deck | 300 lbs/ft$^2$ or AASHTO HS-20 |
| Impact to all moving loads | Per IBC Chapter 16 |

### 11.3.3. Backfill Loads

The lateral earth pressure against vertical faces of structures with horizontal backfill will be computed using the equivalent fluid pressures calculated from:

$$P = kwH$$

Where:

$P$ = unit pressure

$k$ = pressure coefficient

$w$ = unit weight of fill

$H$ = height of fill

For structures free to deflect or rotate about the base the pressure coefficient will be computed from Rankine's theory which is:

$$k_A = \tan^2(45 - \emptyset/2)$$

Where:

$\emptyset$ = angle of internal friction (degrees)

For structures restrained from bending or rotation, the at-rest pressure coefficient will be used:

$$k_o = 1 - \sin\emptyset$$

Coulomb's theory will be used for computing lateral earth pressures on wall surfaces with slopes flatter than 10V:1H or with sloping backfill steeper than 1V:4H.

Surcharge loading will also be taken into account. Where vehicular traffic can run adjacent to the face of a structure, a surcharge loading of 500 lbs/ft$^2$ will be applied.

### 11.3.4. Snow and Ice Loads

Ground snow load will be taken as 120 lbs/ft$^2$ as listed in the ASCE 7-10 Chapter 7 for Talkeetna and based on 200 inches of snow per the Environmental Atlas of Alaska.

Horizontal ice loads will be computed in accordance with USACE EM 1110-2-1612 Ice Engineering (2002). Minimum ice loading will be taken as 5 kips per linear foot.

Ice loading due to spray will also be considered, and structures will be designed to prevent accumulation of ice to the extent practicable. Excessive ice build-up on trashracks, gates, gate guides, support structures and other exposed project elements will be prevented by heating.

### 11.3.5. Crane Loads

The following percentages shall apply to the powerhouse cranes and the power intake cranes. The minimum deflection to span ratio of crane support beams shall be 1:1000.

| | | |
|---|---|---|
| Vertical impact | -25% | static wheel load |
| | -10% | crane capacity, trolley, hook, and lifting beam distributed equally between rails. |
| Longitudinal load | -10% | static wheel loads. |

### 11.3.6. Spillway Deck Loads

| | |
|---|---|
| Areas designated for service | 500 lbs/ft$^2$ |
| Other areas | 200 lbs/ft$^2$ |

### 11.3.7. Hydraulic Loads

All structures shall be designed for full hydrostatic water pressures where applicable, plus full hydrodynamic and uplift forces.

### 11.3.8. Uplift

Uplift pressures shall be taken as equivalent to the full head of water on the foundation or base of the structure where no head differential exists across the structure. Safety factors in accordance with normal conditions will apply. Where a head differential exists across a structure, uplift forces shall be calculated as follows.

For water-retaining concrete structures provided with drainage galleries and drain holes deep into the foundations, uplift shall be considered across the complete rock/concrete interface varying linearly:

1. from $H_1$ at the upstream heel to;
2. $[1/3(H_1-H_2)]+H_2$ at the drains to;
3. $H_2$ at the toe.

Where:

$H_1$ = Static head upstream

$H_2$ = Static head downstream

Safety factors in accordance with normal conditions will apply with drains operative.

Where there are no pressure relief drains, normal uplift shall be assumed to vary linearly from headwater at the upstream face to tailwater at the downstream face. Safety factors in accordance with normal conditions will apply. The latter uplift condition shall also apply for the extreme uplift where drains are to be provided but are assumed to be ineffective in reducing uplift. Safety factors in accordance with extreme conditions will then apply.

Allowable tensile strength at the rock-concrete interface shall be zero. If under earthquake loading conditions a crack is considered to develop at the upstream heel, the uplift pressure shall be taken as equal to the normal distribution as described above over 100 percent of the base area.

Under PMF conditions where cracking at the upstream heel is considered to develop, uplift shall be considered to be equal to full headwater within the full length of the crack, reducing to the values at the line of drains and downstream toe as proportioned above.

Apron and chute slabs and slab walls against rock shall be designed against uplift resulting from sudden changes in water level.

Uplift from centrifugal forces shall be considered where contraction joints occur on the concave floor of chutes.

Toe curve pressures on the interior face of training walls at concave chute surfaces shall be calculated in accordance with Plate 21 of Hydraulic Design of Spillways EM 1110-2-1603 by the U.S. Army Corps of Engineers (USACE 1990).

### 11.3.9. Seismic Loads

#### *11.3.9.1. Seismic Loads*

See other sections.

#### *11.3.9.2. Hydrodynamic Pressure*

The hydrodynamic pressure due to horizontal earthquake on water-retaining surfaces will be computed using the theory of Westergaard for the dynamic change in pressure (lbs/ft$^2$):

$$P = 51.25a(hy)^{1/2}$$

Where:

$h$ = total height of structure (ft.)

$y$ = depth below reservoir surface (ft.)

$a$ = acceleration due to gravity

The distribution of pressure of parabolic; hence, the total force and moment at a section $y$ feet below water level are given by:

$$F = {}^{2}/_{3}\,Py$$

$$M = 0.4Fy$$

### 11.3.10. Temperature and Thermal Loads

Expansion and contraction resulting from temperature changes, moisture changes, creep in component materials, and movement resulting from differential settlement will be combined with other forces and loadings according to the load combinations described.

Temperature data will be taken from the Western Regional Climate Center – Alaska Climate Summaries, Talkeetna WSCMO. The maximum and minimum recorded air temperatures are:

| | |
|---|---|
| Record high | 91°F |
| Record low | -53°F |

### 11.3.11. Wind Load

Wind loads will be computed using the following IBC parameters:

Basic Wind Speed V = TBD mph (50-yr recurrence interval)

Importance Factor for Wind; I = 1.15 for Occupancy Category III & IV for structures

Exposure Category C

## 11.4. Load Cases

Stability requirements for major civil work structures have been developed and are reported in various reference standards. The acceptance criteria to evaluate the stability of a structure considers the uncertainty associated with the analysis (probability of occurrence of certain loading conditions, properties of foundation materials, etc.), as well as an understanding of the classification of the structure and imposed loads. Loading cases and acceptance criteria established by the U.S. Army Corps of Engineers (USACE) will be used to evaluate the performance of structures. A summary of the USACE criteria are presented below.

Stability criteria for the design of new structures and evaluation of existing structures are included in the USACE publication EM 1110-2-2100, "Stability Analysis of Concrete Structures". This publication includes detailed definition of the loading cases and required factors of safety for a variety of concrete structures. The stability requirements are defined as minimum factors of safety for three loading categories (usual, unusual, and extreme) according to the level of site definition and characterization (field investigation and testing of materials). The following loading cases and factors of safety will be considered for the stability analysis and design of structures:

**Table 11-2. USACE Factors of Safety for Concrete Structures**

| Site Information Category | Sliding – Critical Structures | | | Flotation | | | Resultant Location | | |
|---|---|---|---|---|---|---|---|---|---|
| | U | UN | E | U | UN | E | U | UN | E |
| Well Defined | 1.7 | 1.3 | 1.1 | 1.3 | 1.2 | 1.1 | 100% of Base in Compression | 75% of Base in Compression | Resultant Within Base |
| Ordinary | 2.0 | 1.5 | 1.1 | | | | | | |
| Limited | -- | -- | -- | | | | | | |

Notes: U = Usual, UN = Unusual, E = Extreme

Loading cases required to be investigated as part of the stability analysis of concrete structures are also included in USACE EM 1110-2-2100. Seismic stability analyses will include earthquake loadings from the operating basis earthquake (OBE) as appropriate. Ground motion parameters and criteria for the OBE are presented. The loading cases are listed for different types of concrete structures according to their operational requirements. A summary of applicable loading cases is shown below:

**Table 11-3. USACE Load Cases for Concrete Structures**

| Load Case | Loading Description | Classification |
|---|---|---|
| U1 | Normal Pool, All Gates Open | U |
| UN7 | Maintenance Bulkheads in Place | UN |
| UN8 | OBE + Coincident Pool | UN |
| E1* | MCE + Coincident Pool | E |
| E2 | PMF | E |

Notes:

1. Load Case UN7 assumes future bulkhead gates are to be used for maintenance purposes
2. OBE = Operational Basis Earthquake
3. MCE = Maximum Credible Earthquake
4. PMF = Probable Maximum Flood
5. Load Cases U1, UN1, UN8, E1, and E2 are representative of Gravity Dam load cases 2, 3, 5, 6, and 7 according to EM 1110-2-2100

* MCE stability analysis performed with finite element methods. Pseudostaic analysis is inappropriate for this earthquake ground motion.

## 11.5. Minimum Safety Factors and Allowable Stresses

### 11.5.1. Safety Factors against Overturning

#### *11.5.1.1. Structures on Rock Foundations*

In all loading cases except extreme, the resultant of all forces in stability analyses for structures on rock foundations will fall within the middle third of the base width of the structure. No tension will be allowed between the concrete structure and the foundation.

For maximum flood and seismic conditions, the resultant will fall within the base width.

In all loading cases, it will be assumed that only compressive stresses can be transmitted to the foundation and that the portion of the base in tension is ineffective in determining rock foundation pressures. Also, full uplift pressure, if applicable, will be applied to the portion of the base in tension.

#### *11.5.1.2. Structures on Soil*

For normal loading condition, the resultant of all forces in stability analyses for structures on soil will fall within the middle one-sixth of the base width of the structure.

For maximum flood and seismic loading cases, the resultant will fall within the base width and allowable bearing capacity of the foundation will not be exceeded. For other loading cases, the resultant will fall within the middle one-third of the base width.

### 11.5.2. Safety Factors against Sliding

#### *11.5.2.1. Structures on Rock Foundations*

For structures on rock foundations, the shear-friction safety factor against sliding will be computed by the following formula:

$$Q = (fV + sA)/H$$

Where:

$Q$ = Shear-friction safety factor against sliding

$f$ = Friction coefficient

$V$ = Sum of all vertical loads including uplift

$s$ = Unit shearing strength of rock

$A$ = Area of the base under compression

$H$ = Sum of all horizontal forces

Shear-friction safety factor against sliding will not be less than 3.0 for normal operating condition, not less than 2.5 for inspection condition, not less than 2.0 for flood condition, and not less than 1.3 for maximum flood and seismic conditions.

#### *11.5.2.2. Structures on Soil*

For structures on soil, the sliding factor of safety is defined as follows:

$$FS = (V \tan \varphi)/H$$

Where:

$FS$ = Sliding factor of safety

$H$ = Sum of all horizontal forces

$V$ = Sum of all vertical loads including uplift

$\varphi$ = Angle of internal friction for foundation material

The sliding factor of safety will not be less than 2.0 for normal loading condition, 1.2 for maximum flood and seismic loading conditions, and 1.5 for other loading conditions.

### 11.5.3. Safety Factors against Flotation

The safety factor against flotation, defined as the ratio of the sum of all vertical loads (except uplift) over the uplift force, will not be less than 1.3 for normal loading condition, not less than 1.1 for maximum flood condition with water passages empty, and 1.2 for other loading conditions. Safety Factors against uplift will be calculated for all sections of the powerhouse.

### 11.5.4. Allowable Stresses

The allowable stresses for concrete, steel, and other materials will be in accordance with referenced codes and standards.

# 12. RIVER DIVERSION

## 12.1. General

The river diversion facilities, tunnel and cofferdam will be designed to provide approximately a risk of 1 in 10 that the cofferdams will be overtopped and construction work interrupted during the entire period of construction. A detailed risk analysis will be carried out to determine the optimum level of protection during construction. Ultimately, the construction contractor will be responsible for conducting its own risk assessment and for final sizing and design of the river diversion cofferdams.

At a minimum, considering that the main dam is planned as an RCC structure, the sizing of the cofferdams and diversion tunnel will be sufficient to protect the worksite from flooding for events less than the 50-year flood. However, the upstream cofferdam shall be designed so that it can be overtopped, and the sluice through the base of the dam can then be utilized to pass higher flows.

***Hydraulic Criteria***

The diversion works (including tunnel, overtopped upstream cofferdam and sluice) will be designed to pass floods up to and including the routed 50-year flood.

The diversion will be sized to pass ice during winter freeze up and breakup.

The tunnel will be designed to pressurize from the upstream end first in order to avoid hydraulic jumps in the tunnel.

Physical hydraulic model studies may be carried out to ensure satisfactory performance in passing the required flows and in the manner of pressurization.

## 12.2. River Diversion

The river diversion scheme will be sized to pass the 50-year flood. River diversion will be accomplished with a single diversion tunnel to bypass the dam plus a separate sluice through the RCC dam. The upstream cofferdam will have an abutment overflow spillway capable of passing flow to the sluice through the dam. On completion of the river diversion phase, the sluice opening will be plugged and the river diversion tunnel will be modified to become the emergency outlet tunnel. The downstream invert of the tunnel will be set at an elevation either below or at stream bed level such that a hydraulic jump does not form inside the tunnel.

A hydraulic jump is expected to form in the tailrace channel at low flows and in the river downstream at higher flows. Hydraulic jumps can entrain air and cause elevated gas concentrations. The diversion tunnel outlet facilities will be designed to minimize the potential for gas super saturation.

USBR guidelines will be used in designing the structure. A physical hydraulic model and computational fluid dynamics (CFD) studies may be necessary to ensure proper performance of these facilities.

# 13. RCC DAM

## 13.1. Design of Dam

### 13.1.1. Type of Structure

The Watana Dam will be designed as a curved concrete gravity dam constructed using roller-compacted concrete (RCC) methods.

### 13.1.2. Dam Geometry and Layout

The gravity dam axis is circular in plan across the valley, and has straight abutment sections acting as thrust blocks for the curved portion. The final location of the dam axis will be determined based on the final geotechnical field investigation and laboratory testing program.

The following are preliminary design data for the dam and water surface elevations:

***Design Data – Dam***

| | | |
|---|---|---|
| a. | Crest of dam | El. 2065 ft. msl |
| b. | Top of parapet | E1. 2068 ft. msl |
| c. | Crest width | 45 ft. |
| d. | Axis radius for curved portion | 2600 ft. |
| e. | Upstream face above El 1770 | Vertical |
| f. | Spillway crest | E1. 2010 ft. msl |
| g. | Spillway width | 164 ft. |

***Design Data – Water Surface Elevations:***

| | | |
|---|---|---|
| a. | Maximum normal pool | E1. 2050 ft. msl |
| b. | Flood surcharge level (PMF, ref. FR Appendix B4) | E1. 2064.5 ft. msl |
| c. | Tailwater – Normal | El. TBD |
| d. | Tailwater – Flood | El. TBD |
| e. | Sediment (upstream) | El. TBD |

## 13.2. Material Properties

Concrete Material properties for final design will be determined based on field investigations and laboratory testing. The following material properties were estimated from published data for preliminary studies until laboratory test data are available:

### 13.2.1. Water

| | |
|---|---|
| Unit weight | 62.4 pcf |

### 13.2.2. RCC

| | |
|---|---|
| Unit weight | 150 pcf |
| Unconfined static compressive strength (ASTM C39, C172, C31) | $f_c' = 5{,}500$ psi |
| Dynamic compressive strength | 7,150 psi |
| Static direct tensile strength (ASTM C496) | 385 psi |
| Dynamic tensile strength | 580 psi |
| Cohesion | 500 psi |
| Friction angle | 45° |
| Static elastic modulus (ASTM C469) | $3.0 \times 10^6$ psi |
| Dynamic elastic modulus. | $3.9 \times 10^6$ psi |
| Poisson's ratio, | 0.25 |
| Coefficient of thermal expansion | $5.5 \times 10^{-6}$ in/in/°F |
| Diffusivity | 0.045 sq ft / hr |

### 13.2.3. Foundation Rock

Refer to Section 6.5.

## 13.3. Design Loads

The major loads considered in the planning level design are identified in the following sections. The loads will be grouped into loading combinations to assess factors of safety.

### 13.3.1. Dead Load

The dead load of the gravity dam is the weight of mass concrete. The added mass of mechanical equipment, stairs, walkways, railings, etc. should not be considered in the dead load computation. Similarly, the reduced mass of drainage galleries is not to be deducted.

### 13.3.2. External Hydrostatic Loads

A triangular distribution of static water pressure will be assumed acting normal against the upstream face of the dam starting from zero at the water surface. A sediment level at the base of the lowest intake should be considered as additional hydrostatic load.

### 13.3.3. Hydrodynamic Load

The hydrodynamic effect of the reservoir against the dam should be estimated based on the surface area below El. 2050 ft. msl and the orientation of each element. Using the Westergaard method (Zangar 1952), dynamic loads are to be applied to the finite element model as added masses attached to the upstream nodes for the full reservoir condition. The dynamic pressure of water flowing over the spillway crest and downstream slope is neglected.

### 13.3.4. Internal Hydrostatic Loads (Uplift)

Uplift pressure exists within the body of the dam, at the contact plane between the dam and its foundation, and within the foundation below the contact plane. Uplift is assumed to act over 100 percent of the area of any horizontal plane cut through the dam, including the foundation interface plane.

Uplift pressure distribution is assumed to vary linearly from full uplift at the upstream heel to a reduced level at the drain line and then linearly to full tailwater level at the dam toe. The uplift pressure distribution at the line of drains is assumed to exceed the tailwater pressure by one-third of the differential between headwater and tailwater levels.

The uplift pressure diagram is assumed to be the same for normal pool or a raised pool during flooding. The headwater pressure at the heel and tailwater pressure at the toe is adjusted for flooding conditions. The uplift pressure distribution is assumed to be unaffected by earthquake ground motions.

### 13.3.5. Earthquake Loads

Earthquake loads should be selected based on the site-specific seismic assessment.

### 13.3.6. Ice Loads

The method of Monfore and Taylor will be used to estimate anticipated ice pressures when basic data are available. An estimate of ice load to be expected on the face of the dam is assumed to be 10,000 lbs/linear ft. (146 kN/m) of contact between the ice and the dam for an assumed ice depth of 2 feet (0.6 meter) [USBR Monograph 19].

## 13.4. Load Combinations

The dam will be analyzed for the following loading combinations (FERC Engineering Guidelines for the Evaluation of Hydropower Projects-Chapter III Gravity Dams [October 2002]):

1. Usual loading combination: Normal design reservoir elevation with appropriate dead loads, uplift, silt, ice, tailwater, and minimum usual temperatures occurring at that time.

2. Unusual loading combination: Maximum design reservoir elevation with appropriate dead loads, silt, tailwater, uplift, and minimum usual temperatures occurring at that time, if applicable.

3. Extreme loading combination: The usual loading plus effects of the "Maximum Credible Earthquake."

The following load cases were developed from the load combinations identified above:

***Case I – Usual Loading Combination***

- Static Conditions — Normal
- Reservoir at normal maximum pool — El. 2050 ft. msl
- Minimum tailwater — El. 1455 ft. msl
- Silt — El. TBD
- Uplift — drains fully effective

***Case II – Unusual Loading Combination (PMF)***

- Static Conditions — Flood
- Reservoir at PMF pool — El. 2064.5 ft. msl
- Tailwater at maximum Inflow Design Flood condition — El. TBD

- Silt — El. TBD
- Uplift — drains fully effective

***Case III – Extreme Loading Combination (Usual with MCE)***

- Reservoir at normal maximum pool — El. 2050 ft. msl
- Minimum tailwater — El. 1455 ft. msl
- Silt — El. TBD
- Uplift — drains fully effective

Combinations of transitory loads, each of which has only a remote probability of occurrence at any given time, have a negligible probability of simultaneous occurrence and should not be considered as a load combination.

## 13.5. Structural Stability Analysis

### 13.5.1. Static

The sliding stability analysis of the RCC dam will be performed in accordance with procedures contained in EM 1110-2-2200, Gravity Dam Design, and EM 1110-2-2100, Stability Analysis of Concrete Structures, as well as the FERC Guidelines for the Evaluation of Hydropower Projects, Chapter III, Gravity Dams. Static analyses are performed for the specified loading cases using two-dimensional (2D) conventional methods to determine stresses on the dam and foundation as well as the sliding stability factors of safety.

For determining appropriate factors of safety, the RCC dam will be considered in the High Hazard category.

A summary of the stability and stress criteria used for Watana Dam is presented in Table 13-1 below.

**Table 13-1. Stability and Stress Criteria**

| Load Condition | Resultant Location at Base | Minimum Sliding FS | Foundation Bearing Pressure | Concrete Stress | |
|---|---|---|---|---|---|
| | | | | Compressive | Tensile |
| Usual | Middle 1/3 | 3.0 | ≤ allowable | $0.3\ f'_c$ | 0 |
| Unusual | Middle ½ | 2.0 | ≤ allowable | $0.5\ f'_c$ | $0.6\ f'_c{}^{2/3}$ |
| Post Earthquake | Within Base | 1.3 | ≤ 1.33 * allowable | | |

Notes: $f'_c$ is the one-year unconfined compressive strength of concrete. The sliding factors of safety (FS) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions. Post-earthquake analysis will be completed if cracking at the base occurs during the event. Zero tensile stress is allowed across any crack.

### 13.5.2. Dynamic

The curved dam will be analyzed under seismic loading by the response spectrum analysis method, using computer software such as ANSYS.

For final design, finite element analysis using acceleration time histories will be carried out. Non-linear analyses will be performed using LS-DYNA and will include mass in the foundation.

### 13.5.3. Thermal Analysis

Two-dimensional thermal analysis will be performed to assist with development of the final dam geometry. Three-dimensional thermal analysis will then be performed to assess:

- Effects of construction of the abutment blocks earlier than the central dam section;
- Effects of the temperature of the rock foundations – and the value in preventive measures to raise the rock temperature;
- The effects of insulation in the prolonging of the placing season; and,
- The shock loading upon reservoir filling.

## 14. POWERHOUSE

The powerhouse will safely and economically accommodate and protect all the electrical, mechanical and other plant necessary to generate electricity at an installed capacity of 3 x 206 MW = 618 MW.

The powerhouse will include loading, bay, erection bay, electrical and mechanical workshops, stores, offices, control rooms, pantries, toilets, and other facilities normally expected in a large modern powerhouse.

The turbine setting will be such that the turbines can operate free from cavitation problems with a single turbine in operation, and over the full range of expected power heads and discharges.

The powerhouse will have sufficient space and access to enable turbine/generator erection during construction, and to enable maintenance, repair, dismantling and installation of all powerhouse equipment during its operational life.

The powerhouse will be provided with adequate measures to ensure a safe working environment, including fire protection measures.

Estimated tailwater elevations based on 1980s data are stated in the following table (to be refined during site investigations, following additional bathymetric and terrestrial surveying):

**Table 14-1. Estimated Tailwater Levels at Various Discharge Conditions**

| Condition | Flow (cfs) | Tailwater (feet) |
|---|---|---|
| One unit @ 4,600 cfs | 4,600 | 1459.7 |
| Average flow | 8,000 | 1461.1 |
| Three units @ 4,600 cfs | 13,800 | 1462.9 |
| 1 in 25 yr. flood | 71,300 | 1474.2 |
| 1 in 100 yr. flood | 91,300 | 1477.1 |
| 1 in 1,000 yr. flood | 128,000 | 1481.5 |
| 1 in 10,000 yr. flood | 168,000 | 1485.9 |

The powerhouse tailwater deck level and the low-level outlet gatehouse will be set above the 1:10,000 year flood level.

# 15. ROADS AND ACCESS INFRASTRUCTURE

The main elements of the access to the dam site are:

- Transit along the ARRC from the ports of Whittier and Anchorage (depending on the size of the load) to an offloading site at Gold Creek, Chulitna or Cantwell.
- Transit by a new access road along one of three corridors. The three corridors are discussed in the Draft Engineering Feasibility Report.

## 15.1. ARRC Offloading Facilities

The railroad offloading facilities will include spurs with a minimum of two lines each and including a clear container unloading length of 4400 ft.

## 15.2. Access Road to the Site

The access road is discussed in the Draft Engineering Feasibility Report, and will be constructed to the following criteria:

**Table 15-1. Proposed Design Criteria for Access Road to the Site**

| Criterion | Limiting Value |
|---|---|
| Design speed (mph) | 30–50 depending on terrain |
| Minimum radii (ft.) | TBD |
| Max gradient (%) | 12% |
| Desirable gradient (%) | <10% |
| Minimum stopping distance (ft.) | TBD |
| Minimum passing sight distance (ft.) | TBD |
| Maximum Super-elevation | 4 degrees (special loads) |
| Critical grade length | TBD |
| Minimum K crest | 5 |
| Minimum K sag | 8 |
| Lane width (total two way) - (ft.) | 26 |
| Shoulder width (ft.) | 3 |

The standard design load requirement for haulage of regular construction traffic is based on AASHTO HS20-44. A maximum axle weight of 16.5 tons is proposed to account for overloading. The total axle load is therefore 36.5 tons.

Heavy and special loads will be accommodated at times with special escort. These special loads include the delivery and removal of large construction plant equipment such as Caterpillar 777 rock trucks, and the delivery of transformers, penstock cans, generator and turbine parts. Some of the heavy loads will be transported by multi (all steerable) axle, towed low boys, capable of negotiating small radius bends.

To reduce the extent of weather related delays, the road surface will be treated to provide 'all weather' access. The treated surface will require regular maintenance.

## 15.3. Access Around the Site

Access around the Susitna-Watana site will be created to suit the construction and the final configuration of the Project works. To the extent possible, construction access will be modified and rehabilitated at the end of the work to form the permanent access to project facilities.

Roads for construction will extend from the main access road, and will be constructed to various project features including, but not limited to:

- Quarry
- Upstream and downstream cofferdams (on both abutments)
- Powerhouse
- Upstream and downstream diversion tunnel portals
- Construction camp
- Airstrip
- Switchyard
- Water supply and treatment facilities
- Contractors' facilities

The proposed preliminary design criteria for the site roads are listed in Table 15-2.

**Table 15-2. Proposed Preliminary Design Criteria for Roads within Project Works**

| Criterion | Limiting value |
|---|---|
| Design speed | TBD |
| Pavement width | TBD |
| Minimum centreline radii | TBD |
| Stopping sight distance | TBD |
| Maximum gradient | 12 % |
| Minimum gradient | 0.833 % |
| Crest k value | 6 minimum |
| Sag k value | 6 minimum |
| Cross fall | 4 % |
| Junction visibility | TBD |
| Junction radii - minimum | TBD |
| - maximum | TBD |
| Loading | HS 20-44 or equivalent |

## 15.4. Permanent Bridge

A permanent bridge across the Susitna River and located downstream of the dam will be constructed early as an access during construction and for permanent access to each abutment without using the dam crest. The bridge will be a single wide lane bridge with capacity to accommodate a Caterpillar 777 dump truck which has a maximum track and body width of 20 ft.

# 16. POWER FACILITIES

Power facilities and the associated electrical and mechanical balance of plant are described in the Draft Engineering Feasibility Report.

# 17. SITE INFRASTRUCTURE

Construction Camps, contractor's facilities, water, wastewater and the airstrip are described in the Draft Engineering Feasibility Report.