

_README

This directory contains the files for the document, "Plan for Removal of Elwha and Glines Canyon Dams" (Elwha Technical Series PN-95-7) May 1996.

The file "appendix" contains the appendix dividers and the text for appendices a, f, and g. The files 94 ELWHA.EX and 94 GLINES.EX are the SEED examination reports contained in appendix b. The drawings in appendix c are contained in the "drawings" subdirectory to this directory. The schedule in appendix d is located in the "schedule" subdirectory to this directory in both document exchange format (DXF) and AutoCAD 12 format. The photo (TIF file) in the "photo" subdirectory to this directory was prepared for the Elwha River Restoration Project EIS but was not used in the dam removal report (95-7).

D-8470

April 14, 1995

MEMORANDUM

To: Manager, Operation and Structural Safety Group

From: Chris Danley

Subject: Literature Review of and Visit to Elwha Dam - Safety Evaluation
of Existing Dams (SEED) Program - National Park Service (NPS)
- Olympic National Park, Washington

SYNOPSIS

This report for Elwha Dam was prepared during April 1995, and generally reflects studies, analyses, and changes which have occurred up to this time. At the request of the NPS Dams Program Officer, this literature review and site visit was performed as required in NPS-40, Chapter 3, Section A, Paragraph 4, "Acquisition of Dams," to document the current condition of the dam, and to identify any safety of dams, operation and maintenance, and public safety issues related to continued operation of the dams, subsequent to their purchase by the government, and a final decision on disposition of the dam. The site visit to Elwha Dam was performed on September 14, 1994, as was the site visit for Glines Canyon Dam, located about 7 river-miles upstream on the Elwha River. Purchase of both dams by the federal government is anticipated in the near future. One option for subsequent operation would be to remove the dams to restore historic salmon runs on the river and restore physical characteristics of the Elwha River to preconstruction conditions as nearly as practicable. A final decision on disposition of the dam after purchase has not been made.

This evaluation is not intended to interfere with the current regulatory authority of the Federal Energy Regulatory Commission (FERC) nor the rights of the current operator. The purpose of this evaluation is for the possible future owner to become better aware of the conditions and responsibilities they will assume with the acquisition of these projects.

A. General Description

Elwha Dam is located on the Elwha River, about 4 miles north of the northern boundary of Olympic National Park in the northwestern part of the state of Washington, 4 miles upstream from the river's confluence with the Strait of Juan De Fuca. The dam impounds a reservoir (Lake Aldwell) containing

8,100 acre-feet at elevation 195.5. The principal benefit of the reservoir is power

generation, although recreational benefits are available as well. The dam was constructed in 1911-1913.

Elwha Dam, a concrete gravity structure, has a structural height of 108 feet and a crest length of 112 feet at crest elevation 198.0¹. The combined length of the dam crest, powerplant intakes, and spillways, forming the entire water barrier, is approximately 430 feet. The dam contains a vertical upstream face and stepped downstream face. The dam and waterways are shown in photograph 1.

Release facilities at the dam include spillways on both abutments and a powerplant. The left abutment spillway is founded on rock and consists of a gated concrete inlet structure, with flow controlled by four 13.75- by 19-foot radial gates at crest elevation 177.7, and unlined discharge channel with concrete sidewalls. The discharge capacity of the left abutment spillway is 18,700 ft³/s for a reservoir water surface at elevation 200.0.

The right abutment spillway is founded on rock and consists of a gated concrete inlet structure, with flow controlled by five 13.75- by 19-foot radial gates at crest elevation 177.7, a stilling basin, and an outlet canal. The discharge capacity of the right abutment spillway is 23,300 ft³/s for a reservoir water surface at elevation 200.0.

Two powerplants have historically provided commercial power to a local paper mill operation. The powerplants are located side-by-side between the main dam and the left abutment spillway. The intake structure for both powerplants is founded on rock, is about 30 feet high, and contains trashrack protected intakes for two 9.5-foot-diameter penstocks converging to a single 15-foot-diameter penstock, two 30-inch-diameter penstocks, and two 9.5-foot-diameter penstocks. Hydraulically operated slide gates are located at the intakes for each of these penstocks. The two 9.5-foot-diameter penstocks and a single 30-inch-diameter penstock, all 190 feet long, were constructed in 1913. The 15-foot-diameter penstock and related surge tank were installed in 1933. The invert elevation for each of the penstock intakes is 173 feet.

The original powerplant, constructed in 1913, contains horizontal turbine generator units 1 and 2, and one d-c station-service unit. Units 1 and 2 are fed by the two 9.5-foot-diameter penstocks, and are rated at 4,800 horsepower. An Eicher fishscreen was installed on Unit 1 in 1990-1991 as part of a research project. The station service unit, fed by a 30-inch penstock, is rated at 200 kW.

¹All elevations are based on National Geodetic Vertical Datum (mean sea level). Original elevations for the Elwha Project are 8 feet lower than mean sea level.

The second powerplant was constructed in 1922, and contains units 3 and 4, which are vertical turbines connected to short penstocks from the surge tank at the downstream end of the 15-foot-diameter penstock. These units are rated at 5,000 horsepower, and have manually operated butterfly valves in addition to the headgates at the intakes. The combined capacity of the generators is 14.8 MW.

Sluiceway conduits within the main dam section, at invert elevation 119.5, have been plugged and abandoned.

Elwha Dam is operated as a run-of-the-river operation, with the reservoir level being held constant at elevation 187.8 (mean sea level). Maximum generation occurs with 1,800 ft³/s released equally through the four turbines, and any excess flow released over the spillways.

Elwha Dam has been appropriately classified as high hazard, based on the potential for loss of life in the event of a dam failure. An inundation map for failure of Elwha Dam was prepared by Harza Northwest, Inc., in 1992. Peak flows of 159,000 ft³/s at the dam, reducing to 109,000 ft³/s at the Lower Elwha River, resulted from the dam-break simulation. The Lower Elwha Reservation is located near the mouth of the Elwha River.

B. Conclusions

The following conclusions were reached from a review of available records and a site visit:

1. Observations and data review at the time of this evaluation indicate that the dam has been operated and maintained in a generally safe and conscientious manner, although a lack of recent comprehensive dam safety evaluation prevents a complete evaluation of the dam at this time with respect to engineering state-of-the-art. A safety of dams classification code of FAIR is appropriate for Elwha Dam², since no recent seismotectonic analyses have been performed. The FAIR classification indicates that no existing or potential dam safety deficiencies are recognized for normal operating conditions, that all essential elements can be expected to function under normal operating conditions, and that remotely occurring events of magnitudes approaching a probable maximum flood (PMF) or maximum credible earthquake (MCE) will likely (in this case, may) cause a dam safety deficiency. The need for seismotectonic analyses should be considered for the dam, particularly if the anticipated schedule for removal of the dam is delayed due to environmental issues or budget cuts. Periodic inspections of the facilities with respect to operations and maintenance, public safety, and dam safety, are advisable. The frequency of such inspections will probably depend on the type of operation undertaken by

²Based on guidelines in "NPS Dams Inventory User Guide", prepared by Charles Karpowicz, NPS Dams Officer, revised March 1990.

the government. If purchase by the government does occur, a formal dam safety examination should occur within the next few years, due to the high-hazard classification for the dam.

2. Generally adequate preventative measures are in place at the dam and powerplant with respect to safety of the public and operating personnel. Minor additions to the current Public Safety Plan are suggested as a result of this site visit and data review.

3. The current PMF, based on Hydrometeorological Report (HMR) 43, is outdated.

4. Seepage conditions at the dam are similar to previously observed conditions, and presently pose no concerns related to dam safety.

5. Spalling and minimal concrete deterioration exist on the concrete features of the gravity dam, spillways, and power intake. Such deficiencies are not unusual for 80-year-old concrete, and no structural distress was observed on these features.

6. The mechanical features of the spillways are in satisfactory operating condition. Minor repairs for the gates are not planned by the current owner, due to the expected purchase by the government. This is acceptable, as long as all spillway gates continue to be maintained in a fully operable condition, and verified by periodic operating tests, until a final decision is made on disposition of the dam. The current operator maintains the gates in fully operable condition, with full-open tests every five years, and any safety deficiencies are promptly resolved. No deficiencies presently exist that would prevent the proper functioning of the spillways.

7. No deficiencies exist in the structural features of the powerplants that would prevent their continued safe operation for the near future (5 to 10 years). Mechanical features of the powerplant are in generally satisfactory condition. No deficiencies are presently known to exist in the penstocks or surge tank.

8. No additional instrument installations are recommended at the dam at this time.

9. Emergency preparedness considerations at the dam are generally adequate, with the exception of reservoir evacuation capability, which is presently not possible or feasible below the power intake level, due to the deposition of rock and earthfill upstream from the dam in response to the foundation failure in 1912. Although not ideal, this is acceptable for short term operation, because the gravity dam is anchored on its upstream side. Expenditure of funds for investigation of other options for low-level evacuation is not appropriate at this time.

10. The Emergency Action Plan (EAP), and related exercising activities are satisfactory and appropriate for this site.

C. Recommendations

1. Perform formal dam safety examination and prepare report as directed in Federal Guidelines for Dam Safety as soon as possible.

2. Consider the need for seismotectonic analysis of the dam, should the anticipated removal schedule exceed 5-10 years after acquisition of the structure.

3. In order to enhance public safety, give consideration to prohibiting public access to the trail along the left side of the left spillway wall, or as an alternative, improving the trail for public use, to limit the potential for injury, and reduce potential operator liability.

4. Reevaluate public and operational safety considerations for the reservoir and dam remnant areas if dam removal activities are accomplished.

5. Prepare a new PMF, based on HMR 57 inflows, for comparison with the current PMF, should the anticipated removal schedule exceed 5-10 years after acquisition of the structure.

6. Evaluate the interior of the penstocks and surge tank, if operation of these facilities is continued.

7. Monitor the apparent trend of increasing levels in both piezometers in the main gravity section, to assess potential impacts on the safety of the structure.

9. Provide annual updating of the EAP, and continue periodic exercising of the plan after the dam is purchased by the government.

EVALUATION

A. Conditions During Site Visit

The weather during the site visit was cool and rainy, with intermittent rain showers experienced throughout the day.

B. Reservoir Elevations

Reservoir water surface: 187.7

Maximum operating level: 196.0
(Top of active conservation)

Maximum reservoir elevation to date: Unknown

C. Public Safety Considerations

James River Corporation has prepared a Public Safety Plan [1]³ for the Elwha River Hydroelectric Projects, including Glines Canyon Dam and Elwha Dam. As described in the Public Safety Plan, the following public safety measures presently exist at Elwha Dam:

1. Recreational opportunities at the dam (project viewing, hiking, biking, horseback riding) are generally limited to the fenced boardwalk spanning the length of the project features (photos 2 and 3).

2. Boat access to the dam and waterways is prevented by the logboom that spans the reservoir from bank to bank (photo 4).

3. Security fencing and locked gates prevent unauthorized access to restricted areas of the project, including the maintenance shop, spillway winch decks, intake trashracks, spillways, and powerhouse areas downstream from the dam (photos 5, 6, and 7).

4. Warning signs on the left side of the powerhouse wall and at the outer end of the left spillway training wall call attention to the spillways, audible siren, and sudden release of water (photos 8, 9, and 10).

5. Fencing is generally provided at areas where a falling hazard exists (photo 11).

The project owner and operator are to be commended for a strong commitment to public safety, as evidenced by this Public Safety Plan. The following

³Numbers in brackets denote references at end of report.

is provided as a suggested addition to public safety considerations at the project:

- Some fishing occurs in the left abutment tailrace area. Access to this area is by foot, alongside the left wall of the left spillway (photos 12, 13, and 14). Access consists of climbing down rocks and boulders that border the wall, and can be hazardous, particularly during wet conditions. Consideration should be given to prohibiting public access to this trail, or as an alternative, improving the trail for public use, to limit the potential for injury, and reduce potential operator liability.

- Public use areas will change significantly if the dam is removed. Public and operational safety considerations for the reservoir and dam remnant areas should be reevaluated if dam removal occurs.

D. Summary of Dam Safety Activity

Crown Zellerbach Corporation (former owner of the Elwha and Glines Canyon Projects) applied for a Federal Power Commission License in 1968 for the Elwha Hydroelectric Project. This application is still pending because of the controversy surrounding the project related to environmental impacts and other issues. Typically on Federal Energy Regulatory Commission (FERC) regulated projects, FERC requires the project owner to have an independent safety consultant conduct a comprehensive study of the project, termed a "Part 12 Report," every 5 years. Because the Elwha Project is not a licensed project, it has not been subject to Part 12 studies, so periodic comprehensive dam safety evaluations have not occurred at Elwha Dam, as is typical of other projects regulated by FERC. The most recent comprehensive inspection and evaluation report was prepared by R. W. Beck and Associates, Analytical and Consulting Engineers, in 1968. However, annual inspections of the project, which primarily address operational issues, are performed by FERC. Additionally, powerplant operators observe the project features at least once daily during each of the three 8-hour shifts and make weekly, monthly, and quarterly inspections, with results documented.

The following is not intended to be an all-inclusive listing of past designs, modifications, and activities at the dam, but to discuss major activities as they relate to the current safety of the dam.

Review of historical construction data and foundation exploration by private consulting engineers in 1968 [2] indicates that minimal foundation preparation was performed at the time of construction, and that the dam was founded on loose river fill material, by placing concrete under standing water. The pervious foundation blew out in 1912, resulting in the need for foundation stabilization, which was performed in 1913 and 1914.

Stabilization included placing steel piles and the deposition by blasting of large amounts of rock and fill material from the cliffs on the upstream and downstream sides of the dam, then placing layered mats of fir boughs, which were sunk by loading rocks on top of them, in the area upstream from the dam. This provided a short term, substantial reduction in seepage below the dam. Later, increases in seepage resulted in installation of hydraulic fill and a reinforced gunite lining in 1919 for a distance of about 300 feet upstream from the dam. This reduced seepage amounts by about 92 percent. A detailed description of activities prior to 1968 is given in a June 10, 1968 safety investigation report by Shannon and Wilson, Inc [2].

An exploration program conducted for the 1968 consultant foundation assessment [2] indicated that a large (15 feet deep and 40 feet wide) void exists immediately beneath the dam, and that a 50-foot depth of sand, gravel, and boulders underlie the void above bedrock. So, in essence, the main dam is primarily supported by its abutments, and the upstream composite fill and lining serve as the primary water barrier. Only the upper one-fourth of the dam is exposed to the reservoir above the gunite lining. The rockfill acts as a free draining material.

The consultant's safety evaluation [2] concluded that the primary concern related to increasing seepage below the dam is the potential for reduction in power output, and that little potential exists for catastrophic blowout of the foundation, such as occurred in 1912, due to the extensive upstream reservoir blanket of rockfill.

The 1968 consultant safety evaluation [3] concluded that the spillway capacity was insufficient to pass flood inflows that could occur, and that portions of the dam, power intake, and spillway sections would be subject to failure due to loads resulting from flow in excess of spillway capacity. The gravity section, both spillways, and the powerplant intake section were modified in 1980, 1985-86, and 1986-87, to stabilize these features against failure due to overtopping or seismic events. This was accomplished by installing post-tensioned tendons in each spillway pier, two tendons in each bay of the gravity dam (1980), additional tendons in the right (south) spillway piers and gravity section (1986), and an additional nine tendons in intake piers and three additional tendons in piers of the left (north) spillway (1987). After the initial construction in 1980, additional tendons were required based on consultant review, and to meet FERC requirements for stability.

E. Hydrology

The drainage basin behind Elwha Dam is about 315 square miles. Operated as a run-of-the-river structure, release facilities at the dam are required to pass flood inflows, with little surcharge storage available to reduce peak outflows. The dam was modified in 1980-1987 by placing post-tensioned tendons in the main gravity section, power intake, intake, and spillway

piers, to increase stability against overtopping or sliding failure during PMF peak inflow of 180,000 ft³/s, at maximum reservoir elevation 212.5, with all spillway gates open.

The current PMF was computed based on rainfall values in HMR 43. This report has been superseded by the recent completion of HMR 57, which covers the Pacific Northwest. In order to maintain currency with state-of-the-art engineering practice, a new PMF should be computed, based on HMR 57 inflows, for comparison with the current PMF. If the resulting peak inflow greatly exceeds the current predicted peak, it is possible that safety factors with the existing tendons could be exceeded. If this was the case, the need for additional modifications would have to be evaluated, based on the expected remaining life of the project.

F. Dam and Appurtenances

1. Seepage. During this onsite evaluation, the level of the tailwater pool downstream from the main gravity section was similar to previous observations by FERC and others. Seepage below the main gravity section is expected due to the unconventional repairs to the dam following the 1912 blowout of the foundation. A private consultant measured flows of 70 ft³/s in 1972 [4]. The 1994 FERC prelicense inspection report [4] recommended that a staff gage be installed on the canyon wall to measure pool level as a relative indicator of seepage flow (photo 15). During the preparation of this report, it was learned that a staff gage has been installed per this recommendation, and that consistent readings had been recorded at the gage since the fall of 1994.

The tailwater pool has occasionally become cloudy, assumedly from loss of fines from the foundation with seepage. This does not pose a major concern, due to the massive upstream rockfill buttress. The 1968 Shannon and Wilson, Inc., report [2] indicates that even in the event of a major earthquake, there is little likelihood that water would escape catastrophically from the foundation, as it did in 1912, provided that the concrete gravity dam remains intact. Although no seepage/piping related Safety of Dams concerns are noted, past repairs of foundation conditions do not meet state-of-the-art procedures and practices, due to the makeshift nature of the repairs, and perceived lack of engineering evaluations of conditions prior to the repairs.

A V-notch weir measures seepage from the left abutment (photo 16). Approximately 5 gal/min was passing over this weir during this site visit. Seepage measured at this weir reportedly is constant throughout the year, which is as would be expected with consistent reservoir levels.

Seepage conditions at the dam are similar to previously observed conditions, and presently pose no concerns related to dam safety.

2. Dam. The safety inspection report completed in 1968 included extraction of cores from the concrete dam, and compressive testing of the cores. Test results indicated that zones of low strength concrete exist, with compressive strengths in the 2,000-2,500 lb/in² range.

The concrete in the dam exhibits deterioration and spalling, expected for a dam of its age, and given the construction practices at the time it was constructed (photo 17). No areas of structural distress were observed. The downstream ends of the plugged sluiceway conduits were examined during the 1994 FERC prelicense inspection [4]. During that inspection, concrete was observed to be generally sound on the surface of the sluiceway conduits, with one spalled area observed.

The installation of post-tensioned tendons in 1980, 1986, and 1987 enhanced the stability of the gravity dam. The 1968 Shannon and Wilson evaluation report [2] indicated that because of the modifications performed after foundation failure in 1912, the dam is essentially a rockfill dam with an impervious upstream lining (gunite), with the downstream slope of the rockfill replaced by the concrete dam. No deficiencies related to the concrete gravity dam or its ability to serve with the upstream rockfill and lining as a water barrier are known to exist at this time, although current state-of-the-art seismotectonic studies have not been performed, and so the ability of the dam to withstand earthquake shaking is not accurately known. Seismic analyses should be considered if planned removal of the dam goes beyond an anticipated 5-10 year period.

3. Spillways

a. Structural features. Spalling and surface deterioration of the concrete was observed on both spillways (photos 18 and 19). This deterioration due to age is not unusual, and does not threaten the continued safe operation of the spillways. No areas of structural distress were observed.

b. Mechanical features. The gates reportedly operate in a satisfactory manner [4]. Minor repairs to gates No. 1 and 7 were identified in the 1994 FERC prelicense inspection report [4]. No plans to make these minor repairs are being made, pending final disposition of the dam. This examiner agrees with this strategy, as long as all spillway gates are maintained in a fully operable condition, and verified by periodic operating tests, until a final disposition is made.

No deficiencies presently exist that would prevent the proper functioning of the spillways.

4. Powerplants

a. Structural features. No deficiencies exist in the structural features of the powerplants (photos 20 and 21) that would prevent their continued safe operation for the near future (5 to 10 years).

b. Mechanical features. Mechanical features of the powerplant are in generally satisfactory condition. No deficiencies are presently known to exist in the penstocks or surge tank; however, as indicated in a 1994 assessment of interim operation of the Elwha and Glines Canyon powerplants prior to final disposition, by personnel from Reclamation's Pacific Northwest Region, one option for interim operation is to discontinue interior inspections of these features for the expected remaining life of the plants, due to the cost required for confined space entry requirements. This limits the amount of time that the powerplants can continue to run safely before they must be shut down, or inspected and probably repaired. If the plant is shut down, no inspections would be necessary. The need for continued interior inspections of these features for the expected remaining life of the plant should be evaluated based on the interim operation scheme for the plant.

Except for potentially affecting reservoir evacuation capability, the failure of penstocks or other features of the powerplant would not threaten the safety of the dam, although this could pose a threat to the safety of operating personnel. However, the condition of the penstocks and surge tank should be verified until a final decision on disposition of the dam is made.

5. Instrumentation. Instrumentation at Elwha Dam consists of two open-tube piezometers installed in 1968 and 1986, respectively, in the gravity dam, and a seepage measurement weir downstream from the left abutment. A third piezometer, installed in 1980, subsequently became plugged, and is abandoned.

A review of piezometer plots during the 1994 FERC prelicense inspection revealed an apparent trend of increasing levels in both piezometers, by 3 to 5 feet, since 1987. This may indicate deterioration of the upstream earth and rockfill blanketing with time. Although the existence of post-tensioned tendons in the gravity section will prevent structural failure due to increasing uplift pressure beneath the section, this trend of increasing uplift should be closely monitored.

With the exception of the recommended staff gage on the canyon wall downstream from the main gravity section for relative seepage measurement, as mentioned previously, no additional instrument installations are recommended at the dam at this time.

G. Emergency Preparedness

1. Access to the site. The EAP lists primary and alternate routes to the dam, including access by foot, four-wheel drive vehicle, floatplane,

helicopter, and boat. Additionally, organizations to be contacted to obtain alternate methods of transportation are listed in the EAP, with their telephone numbers. Access to the dam is adequate.

2. Communication system. Communications at Elwha Dam, listed in the EAP, include commercial telephone service, the Port Angeles intra-mill telephone system, a dedicated telephone system on a "hot line" circuit from the powerhouse to Peninsula Communications (PenCom), UHF radio communications (providing a direct page to PenCom, communications to Glines Canyon Dam and The Daishowa America mill, communication to mobile radios, and communication with the mill paging system with interface to the telephone system), and VHF radio communications (providing communication to Glines Canyon Dam and the Daishowa America Mill, communication to mobile radios, communication to portable radios, and communication to the Clallam County Emergency Operations Center). These communications options, and directions for accessing the various communications systems in an emergency, are listed in the EAP. The communication system is adequate.

3. Warning system. The only public warning system at the dam is a siren that sounds prior to making releases via the spillway gates. The siren was test operated during this site evaluation, and operated satisfactorily.

4. Auxiliary power. The EAP lists six sources of power for the spillway gates. The first source is the No. 2 exciter, connected to the small station-service turbine. The auxiliary power sources are adequate.

5. Remote operation. The left spillway hoist can be operated remotely from the control room, although the right spillway hoist is operated with local controls at the gates. Due to adequate onsite attendance, this scheme is adequate.

6. Security of the site. Security at the dam consists of locked gates, fencing, and doors preventing access to restricted areas of the dam and powerplant. The control room at Elwha Dam is manned continuously by an operator who controls both Elwha Dam and Glines Canyon Dam. No problems related to vandalism were reported by operating personnel. Security of the site is adequate.

7. Reservoir evacuation capability. Presently, reservoir evacuation is available to elevation 173, the penstock invert elevation, which is 15 feet below the normal operating level. The reservoir depth at the upstream face of the dam is quite shallow, due to the rock, earthfill, and gunite lining that were deposited in response to the foundation failure of 1912. Sluiceway conduits at invert elevation 119.5, which are abandoned and plugged with concrete, are not available for reactivation because their intakes are now submerged by about 50 feet of earthfill and rock. Additionally, evacuation to the spillway crests at elevation 177.7 is available by opening the spillway gates. No other options for low-level

evacuation exist because the spillways and power intake are founded on bedrock.

It is sound engineering practice to provide for evacuation of the reservoir to low levels, for inspection or emergency drainage purposes. Although it would be preferable for Elwha Dam to have some means to drain the reservoir to view the upstream earthfill and rock buttress, and for emergency purposes, no practical means exist to allow for this, and no expenditures to investigate low-level evacuation are warranted. This is even more true now that the dams are being considered for removal. The lack of low-level evacuation capability is therefore acceptable in this case.

8. Operating instructions and Emergency Action Plan. An EAP for Glines Canyon and Elwha Dams was revised in 1994. Tabletop exercise tests of the EAP are conducted quarterly among operating personnel, and annually with other county, NPS, and local emergency personnel. The most recent annual test was conducted in November 1994, and consisted of a simulated onsite emergency, and the resulting communication to appropriate authorities. Based on subsequent conversations with NPS personnel, this exercise was quite useful in identifying potential problems in the communications facilities, primarily with portable communications. Deficiencies identified during this exercise will be corrected in the near future. The EAP contains adequate instructions to perform the required operations and activities in an emergency.

Operating personnel are well versed in procedures for operating equipment at the dam. Full instructional information is maintained at the powerplant. Operating instructions and EAP activities are adequate.

Attachments 7

List of Participants

References

Current National Park Service Dams and Related Floodplain Inventory
Report for Elwha Dam (revised by examiner)

Photograph Pages

Figure 1 - General Layout and Sections⁴

Figure 2 - Cross Section of Dam

Figure 3 - Contours of Reservoir and River Bottom

Review Certification

⁴All figures reproduced from Periodic Safety Inspection Report, Elwha Hydroelectric Project, by R.W. Beck and Associates for Crown Zellerbach Corporation, dated October 1968.

This report has been reviewed and is believed to be in accordance with the project requirements, standards of the profession, and Reclamation policy.

Reviewer: _____ Date: _____
Darian Ingram

LIST OF PARTICIPANTS

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REFERENCES

- [1] Public Safety Plan, Elwha River Hydroelectric Projects, James River II Inc., revised August 20, 1993.
- [2] Report on Safety Investigation of Main Dam Section, Elwha Hydroelectric Project, to R. W. Beck and Associates, Seattle, by Shannon and Wilson, Inc., Seattle, Washington, dated June 10, 1968.
- [3] Periodic Safety Inspection Report, Elwha Hydroelectric Project, by R. W. Beck and Associates for Crown Zellerbach Corporation, dated October 1968.
- [4] Federal Energy Regulatory Commission, Portland Regional Office, Memorandum to Files, Preliminary Inspection, Elwha Project, FERC No. 2683, James River Paper Company, Applicant, dated May 10, 1994.

ADDITIONAL REFERENCES

- Phase I Inspection Report, National Dam Safety Program, Elwha River Basin, Elwha Dam, Port Angeles, Washington, WA-242, Seattle District, U.S. Army Corps of Engineers, dated August 1978.
- The Elwha Report, Restoration of the Elwha River Ecosystem and Native Anadromous Fisheries, A Report Submitted Pursuant to Public Law 102-495, dated January 1994.
- Elwha Dam, Port Angeles, Washington, Post-tensioning Tendon Installation, Summary of 1986-1987 modifications by Harza Engineering Company.
- Elwha Dam, Washington, Post-tensioning Tendon Installation, Construction Report, Prepared for Crown Zellerbach Corporation, Port Angeles Mill, Port Angeles, Washington, by CH2M Hill Northwest, Inc., Portland, Oregon, dated May 1986.
- Emergency Action Plan, Elwha Dam and Glines Canyon Dam, FERC Project Numbers 2683 and 588, James River Paper Company, Inc., Port Angeles Mill, Ediz Hook, Port Angeles, Washington, for Calendar Year 1993, submitted December 29, 1992.

Photo 1. Elwha Dam - General view, facing upstream, of right abutment spillway, main gravity section, power intake and powerplant, and left abutment spillway.9-14-94

Photo 2. Elwha Dam - Security fencing protects public access boardwalk on crest of dam, spillways, and power intake. 9-14-94

Photo 3. Elwha Dam - View of upstream side of right abutment spillway, gravity section, and power intakes. Note protective fencing along entire crest. 9-14-94

Photo 4. Elwha Dam - Logboom spans entire width of reservoir upstream from dam. 9-14-94

Photo 5. Elwha Dam - Locked gate prevents public access
to right spillway hoist area. 9-14-94

Photo 6. Elwha Dam - Locked gate prevents public access
to powerplant from crest of dam. 9-14-94

Photo 7. Elwha Dam -
Protective signs at
entrance to
powerplant.

9-14-94

Photo 8. Elwha Dam - Warning sign at left end of dam crest.

9-14-94

Photo 9. Elwha Dam - Warning sign at downstream left end
of left spillway chute wall, near tailrace fishing area.
9-14-94

Photo 10. Elwha Dam - Warning sign at left wall of
powerplant.
9-14-94

Photo 11. Elwha Dam - Extension of dam crest fencing on
the right abutment protects against falling hazard.9-14-94

Photo 12. Elwha Dam - Access from crest of dam to tailrace
fishing area by foot. 9-14-94

Photo 13. Elwha Dam - Foot access to tailrace fishing area delineated by arrow. Note left abutment seepage measuring weir at bottom of photograph, shown in photograph 16.

9-14-94

Photo 14. Elwha Dam - Closeup view of access path to tailrace fishing area.

9-14-94

Photo 15. Elwha Dam - Tailrace downstream from gravity dam, fed by foundation seepage. Cascading flow at right side of photograph is from right spillway gate leakage.
9-14-94

Photo 16. Elwha Dam - Seepage measurement weir on left abutment.
9-14-94

Photo 17. Elwha Dam - Downstream face of gravity dam.
9-14-94

Photo 18. Elwha Dam - Typical pier and gate of right
spillway. 9-14-94

Photo 19. Elwha Dam - Left spillway gate structure and chute. 9-14-94

Photo 20. Elwha Dam - Penstocks, surge tank, and powerplant structures. 9-14-94

Photo 21. Elwha Dam - Typical condition of concrete in power intake. Note minor spalling and weathering.9-14-94

D-8470

April 14, 1995

MEMORANDUM

To: Manager, Operation and Structural Safety Group

From: Chris Danley

Subject: Literature Review of and Visit to Glines Canyon Dam -
Safety Evaluation of Existing Dams (SEED)
Program - National Park Service (NPS) - Olympic
National Park, Washington

SYNOPSIS OF EXAMINATION

This report for Glines Canyon Dam was prepared during April 1995, and generally reflects studies, analyses, or changes up to this date. The site visit of Glines Canyon Dam was performed on September 14, 1994, as was the site visit for Elwha Dam, located about 7 river-miles downstream on the Elwha River. Purchase of both dams by the federal government is anticipated in the near future. A final decision on disposition of the dams after purchase has not been made. One option would be to remove the dams to restore historic salmon runs on the river and restore physical characteristics of the Elwha River to predam conditions as nearly as practicable. At the request of the NPS Dams Program Officer, this literature review and site visit was performed as required in NPS Special Directive 87-4, Dams and Appurtenant Works and NPS-40, Chapter 3, Section A, Paragraph 4, "Acquisition of Dams." The purpose of this activity is to determine the current condition of the dams, and to identify any safety of dams, operation and maintenance, and public safety issues related to operation of the dams, prior to their purchase by the government, and a final decision on disposition of the dams.

This evaluation is not intended to interfere with the current regulatory authority of the Federal Energy Regulatory Commission (FERC) nor the rights of the current operator. The purpose of this evaluation is for the possible future owner to become better aware

of the conditions and responsibilities they will assume with the acquisition of these projects.

A. General Description

Glines Canyon Dam is located on the Elwha River in Olympic National Park in northwest Washington, and impounds a reservoir (Lake Mills) containing 40,500 acre-feet at normal water surface elevation 590.3. The principal benefit of the reservoir is commercial power production, although recreational benefits are also provided. The dam is a concrete thin arch dam with a maximum structural height of 210 feet and a crest length of 150 feet at crest elevation 590.3¹. The dam was completed in 1927. The arch section is 4 feet wide at the crest, and approximately 50 feet wide at the deepest point of the base. The upper 40 feet of the arch dam is reinforced at the abutments. Piers and a walkway section atop the arch were removed during modifications in 1969, due to concerns related to overtopping of the dam. The arch dam serves as an overflow spillway with a discharge capacity of 86,000 ft³/s at reservoir elevation 611.8. The right end of the main arch section connects to a mass concrete thrust block on the right abutment, adjoining a shallow concrete gravity section, concrete wall, and an 185-foot-long earthfill auxiliary dike at crest elevation 612.3 on the extreme right abutment. The reinforced concrete wall is 55 feet long, and 3 feet thick. The gravity section is 55 feet long, and contains a 0.7H:1V downstream face batter below approximate elevation 593, with an 18-inch-thick concrete wall above this elevation. The wall and gravity section contain a 6-foot-wide reinforced concrete deck at elevation 600.3, which is 12 inches thick. The wall is reinforced for its full height, and the upper portion of the gravity section is reinforced. The right abutment auxiliary dike, modified in 1969, has an 8-foot-wide crest width, and 1H:1V slopes on the upstream and downstream faces. The upstream face contains riprap slope protection below approximate elevation 595, and grass cover above, with grouted riprap on the right end.

A 260-foot-long earthfill freeboard dike, which was constructed in 1973, is located to the left of the spillway on the left abutment. This embankment section contains an 8-foot-wide crest at elevation 612.3, and 1-1/2H:1V side slopes. Both slopes are protected by grass cover.

¹All elevations are mean sea level datum.

Release facilities at the dam include the spillway and powerplant. The spillway, located on the left abutment, continues along the curve of the arch in plan view, and consists of a gated crest at elevation 570.33; flow is controlled by five 20- by 20-foot radial gates. Grooves for stoplog placement are located upstream from the gates. Releases then pass into a concrete chute, and unlined plunge pool at the base of the dam. A 120-foot-long bridge deck spans the length of the spillway crest at elevation 600.3. The discharge capacity of the spillway is 63,000 ft³/s at reservoir water surface elevation 611.8.

A sluiceway opening exists within the arch section at centerline elevation 450.3. The sluiceway consists of a circular intake with a trashrack of hemispherical shape; a 6-foot-long section within the arch concrete, transitioning from 11 feet diameter to 5 feet diameter; a 5-foot-diameter butterfly valve; and a 6-foot 6-inch-diameter downstream steel liner to the downstream face of the arch. The butterfly valve operator is accessed by a ladder housed in a square concrete shaft on the downstream face of the dam, and horizontal adit.

The powerplant is located about 500 feet downstream from the left abutment of the dam, and is fed via an 11- to 13.5-foot-diameter gunite-lined circular tunnel within the abutment, a 10-foot-diameter pressure pipe, and an 11-foot, 7-inch-diameter penstock. The total length of the pressure tunnel and penstock is 857 feet. The power tunnel alignment contains a trashrack structure at the inlet at invert elevation 501.4, located about 200 feet into the reservoir from the left abutment shoreline; a 7- by 12-foot guard gate (at sill elevation 503.3) 190 feet downstream from the inlet; and a hydraulically operated 8-1/2-foot-diameter butterfly valve, which serves as a shutoff valve for the turbine. A 140-foot-high surge tank is located along the pipe alignment near the downstream tunnel portal. Releases are made through a 17,500-hp Francis Turbine. The discharge capacity of the turbine is 1,150 ft³/s when generating at reservoir water surface elevation 590.33.

Glines Canyon Dam has been appropriately classified as high hazard, based on the potential for loss of life in the event of a dam failure. An inundation map for failure of Glines Canyon Dam was prepared by Harza Northwest, Inc., in 1992. Peak flows of 650,000 ft³/s at the dam, reducing to 350,000 ft³/s at the Lower Elwha River (assuming overtopping failure of Elwha Dam), resulted from the dam break

simulation. The Lower Elwha Reservation is located near the mouth of the Elwha River.

B. Conclusions

The following conclusions were reached from a review of available data and a site visit.

1. Observations and data review at the time of this onsite examination indicate that the dam has been operated and maintained in a generally safe and conscientious manner, with general adherence to engineering state-of-the-art and proper maintenance. A safety of dams classification code of SATISFACTORY is appropriate for Glines Canyon Dam². The SATISFACTORY code indicates that no existing or potential dam safety deficiencies are recognized, that all essential elements can be expected to function under all conditions, including such events as the probable maximum flood (PMF) or maximum credible earthquake (MCE), which have only a remote chance of occurring. Continued periodic inspections of the facilities with respect to operations and maintenance, and dam safety, are advisable. The frequency of such inspections will probably depend on what type of interim operation is undertaken by the government. The next Part 12 inspection would be expected to occur in 1997, based on the current five year inspection frequency. If the dam is acquired by the government, a formal dam safety examination should occur within the next few years.

2. Generally adequate preventative measures are in place at the dam and powerplant with respect to safety of the public and operating personnel at the damsite. Suggested additions to the Public Safety Plan are made in this report to further improve public safety considerations at the project.

3. The current probable maximum flood (PMF), based on HMR 43, can be passed with flows through the spillway and over the arch dam crest; the maximum reservoir level would be at elevation 611.8 feet. The PMF should be recomputed, based on HMR 57 inflows, and compared with the current PMF, should the anticipated removal schedule exceed 5-10 years after acquisition of the structure. If the resulting peak inflow greatly exceeded the current peak, it is possible that safety factors for the existing anchor installations could be exceeded, or

²Based on guidelines in "NPS Dams Inventory User Guide," prepared by Charles Karpowicz, NPS Dams Officer, revised March 1990.

overtopping of the abutment dikes could occur. If this was the case, the need for additional modifications would have to be evaluated, based on the expected remaining life of the project.

4. No significant concerns related to seepage have been experienced at the dam to date. No potential dam safety deficiencies exist related to seepage.

5. No potential dam safety deficiencies exist related to the arch, gravity section, thrust block, or embankment dikes. These features have been determined by engineering analyses to be stable against all loading combinations, including normal, seismic, and PMF loadings. No deficiencies that could affect the structural integrity of these features were observed during this site visit.

6. No deficiencies presently exist that would prevent the proper functioning of the spillways. All spillway gates should continue to be maintained in a fully operable condition, and verified by periodic operating tests, until the dam is deactivated. No areas of cracking, concrete deterioration, or structural distress were reported in recent inspection reports, or observed during this examination.

7. No deficiencies exist in the structural features of the powerhouse that would prevent their continued safe operation for the near future (5 to 10 years). Mechanical features of the powerplant are in generally satisfactory condition. No deficiencies are presently known to exist in the penstocks or surge tank. The condition of the penstocks and surge tank should be verified until a final decision on disposition of the dam is made.

8. Existing instrumentation data indicates satisfactory performance of the dam.

9. Emergency preparedness considerations at the dam are generally adequate, with the exception of reservoir evacuation capability and access to the site. Reservoir evacuation capability is presently not possible or feasible below the power intake level, due to the operator's reluctance to attempt releases from the low-level sluiceway. Although not ideal, this is acceptable for short term operation, and expenditure of funds for investigation of options for low-level evacuation is not appropriate at this time.

The only access road to the dam recently experienced a partial washout, and complete washout of the roadway is possible in the

future. Adequate access is required to allow for prompt response to emergency situations at the dam. Due to the roadway instability, it is important that all alternate access routes listed in the EAP be verified periodically, and well understood by operating personnel.

10. The Emergency Action Plan (EAP) contains adequate instructions to perform the required operations and activities in an emergency. Updating and exercising of the EAP should continue prior to and after the anticipated acquisition of the dam by the government. Operating instructions and EAP activities are adequate.

C. Recommendations

1. Replace the missing cross member on the handrail atop the right abutment gravity section.

2. Install an additional sign or signs in the occasional visitor use area of high cliffs downstream from the right abutment, to warn people that enter this area of the possible dangers, and the consequences, of approaching the cliffs. The warning signs should include wording such as EXTREME DANGER PAST THIS POINT - 200 FOOT CLIFF - VEGETATION OBSCURES DROPOFF.

3. Install a more secure railing on top of the arch dam crest for use by surveyors, because of the potential for hazardous falls from the crest; Survey personnel should be tied off with safety lines to ensure their safety. As an alternative, periodic surveys could be performed by boat.

4. Reevaluate public and operational safety considerations for the reservoir and dam remnant areas, if dam removal activities are accomplished.

5. Compute a PMF based on Hydrometeorological Report No. 57 (HMR 57) inflows, for comparison with the current PMF, should the anticipated removal schedule exceed 5-10 years after acquisition of the structure.

6. Continue to maintain all spillway gates in a fully operable condition, verified by periodic operating tests, until a final decision on disposition of the dam is made.

7. Continue to evaluate the interior of the penstocks and surge tank, if operation of these facilities is continued.

8. Continue to update the Emergency Action Plan and periodically exercise the plan prior to and after the dam is purchased by the government.

9. Determine actions necessary to ensure adequate vehicle access to the dam during adverse conditions.

EXAMINATION

A. Conditions During Field Examination

The weather during the onsite examination was cool and rainy, with intermittent rain showers experienced throughout the day.

B. Reservoir Elevations

Reservoir water surface: 589.6

Maximum operating level: 590.3
(Top of active conservation)

Maximum reservoir elevation to date: 593.3 [1]³ (December 22, 1936)

C. Public Safety Considerations

James River Corporation has prepared a Public Safety Plan [2] for the Elwha River Hydroelectric Projects, including Glines Canyon Dam and Elwha Dam. As described in the Public Safety Plan, the following public safety measures presently exist at Glines Canyon Dam:

1. A siren is sounded at the dam to warn people immediately downstream in advance of spillway gate openings, and increasing riverflow. Warning signs call attention to the spillway, and the significance of the siren sounding.

2. A floating logboom spans the reservoir from bank to bank, about 300 feet upstream from the dam (photo 2).

3. An 8-foot-high fence and locked gate at the powerplant discourage entrance to the canyon from the left riverbank.

4. A locked gate with "Authorized Personnel Only" signage prevents unauthorized entry to the spillway hoist deck (photo 3).

5. The power intake gatehouse is locked with "No Admittance" signage. The bridge walkway leading to the winch house is a public access area, protected with standard pipe handrailings (photo 4).

³Numbers in brackets denote references at end of report.

6. Posted warning signs at the right abutment parking area and thrust block warn of high bluffs along the river canyon, which are obscured by brush and undergrowth.

7. Public access is prevented beyond the center of the right abutment gravity section by an extended security fence, locked gate, and "No Admittance" signage (photo 5).

8. Security fencing below the thrust block prevents access to hazardous bluffs in this area.

9. Public access to the powerhouse area is limited to foot traffic by a locked vehicle gate on the access road. A sign at the powerhouse calls attention to the spillway releases, and the significance of the siren sounding.

10. The service road to the transformer and switchyard is blocked by a locked vehicle gate. The switchyard is enclosed behind a security fence and locked gate. The stairway down to the powerhouse from the switchyard contains a locked security gate (photos 6, 7, and 8).

Additionally, the left abutment cliffs downstream from the dam contain a security fence to prevent falls (photo 9). The project owner and operator are to be commended for a strong commitment to public safety, as evidenced by this Public Safety Plan. The following is provided as suggested additions to public safety considerations at the project:

1. The handrail atop the right abutment gravity section is missing one section of lower cross member, on the downstream side, immediately to the left of the locked gate (photo 5). The missing member should be replaced.

2. The area downstream from the dam on the right abutment contains steep cliffs with about a 200-foot vertical drop. Operating personnel indicated that visitor use of an open area near the cliff dropoff is common, for picnicking or viewing the dam (photo 10). The dangerous dropoff is not fenced, and is obscured by vegetation in some areas. Although the area of danger is not readily visible from the parking area, and a warning sign exists along the foot access path to this area, an additional sign or signs should be installed to warn people that enter this area of the possible dangers, and the consequences, of approaching the cliffs. The

warning signs should include wording such as EXTREME DANGER PAST THIS POINT - 200 FOOT CLIFF - VEGETATION OBSCURES DROPOFF.

3. Since the walkway and piers were removed from the arch, a cable way was installed on top of the dam crest to facilitate periodic monument surveys (photo 11). Because of the potential for hazardous falls from the crest, a more secure railing should be installed, or the surveys should be performed by boat. Survey personnel should be tied off with safety lines to ensure their safety.

4. Employee access to docks near the left abutment of the dam is by open planking bridges (photo 12). No handrail exists on the walkway, and the potential for slips and falls exists on the planking. The walkway surface should be monitored for areas of deterioration and poor footing.

5. Public use areas will change significantly if the dam is removed. Public and operational safety considerations for the reservoir and dam remnant areas should be reevaluated if dam removal occurs.

D. Summary of Dam Safety Activity

The following is not intended to be an all-inclusive listing of past designs, modifications, and activities at the dam, but to discuss major activities as they relate to the current safety of the dam:

The project was first licensed by the Federal Power Commission for a period of 50 years in 1926. The Federal Energy Regulatory Commission (FERC) has issued annual licenses for the Glines Canyon Project since 1976. FERC inspects the project annually.

FERC requires the project owner to have an independent safety consultant conduct a comprehensive study of the project, termed a "Part 12 Report", every 5 years, and requires that the owner correct major deficiencies identified within 5 years. The most recent Part 12 Report was completed in 1992 by Harza, Inc. [3].

Modifications have occurred to the right abutment gravity section in 1969, 1973, and 1989. In 1969, 12 post-tensioned tendons were installed in the gravity section.

Modifications occurred in 1973 to safely pass the PMF. The vertical wall portion of the gravity section was anchored to the foundation

by four 45-foot-long prestressed anchors. The right abutment dike was raised to elevation 612.3, and the left abutment freeboard dike was constructed.

As a result of recommendations in the 1987 Part 12 Report, the gravity section and spillway piers were further strengthened against overtopping by the PMF, with the installation of post-tensioned tendons in 1989.

Powerplant operators observe the project features at least once daily during each of the three 8-hour shifts, and make weekly, monthly, and quarterly inspections, with results documented and maintained onsite.

E. Hydrology

The dam has been modified to safely operate during the PMF, with a peak inflow of 149,000 ft³/s, and a maximum reservoir elevation of 611.8 feet. This results in about 0.5-foot of freeboard to the crest of the abutment dikes, which appears adequate for the short duration which would occur. The PMF inflow was recalculated in 1980 based on rainfall values in Hydrometeorological Report No. 43 (HMR 43). The resulting peak inflow was equal to the peak inflow obtained from the previous 1968 flood study. However, HMR 43 has been superseded by the recent completion of HMR 57, which covers the Pacific Northwest. In order to maintain currency with state-of-the-art engineering practice, a PMF should be computed, based on HMR 57 inflows, for comparison with the current PMF. If the resulting peak inflow greatly exceeds the current peak, it is possible that safety factors for the existing anchor installations could be exceeded, or overtopping of the abutment dikes could occur. If this was the case, the need for additional modifications would have to be evaluated, based on the expected remaining life of the project.

F. Dam and Appurtenances

1. Seepage. No significant concerns related to seepage have been experienced at the dam to date. During this examination, no seepage was observed on the downstream face of the dam, and no foundation seepage was observed in the vicinity of the arch. Close inspection of the downstream face in 1991 [4] indicated that minor seepage exists at two horizontal construction joints. The seepage has been partially sealed off by calcite deposits.

Three foundation drain holes exit on the downstream face of the gravity section, and typically flow a slight amount or not at all. Right abutment foundation seepage is monitored at a V-notch weir located downstream from the gravity section (photo 13). A trickle of flow was passing over the weir during this examination, with the reservoir full. This is similar to past performance. Another drain, within the thrust block, was flowing during this examination (photo 14).

No potential dam safety deficiencies exist related to seepage.

2. Dam. The dam is constructed in a narrow gorge composed of tertiary graywacke. The rock contains steep jointing, almost at right angles to the direction of riverflow. All of the concrete structures are founded on the graywacke, which has a high unconfined compressive strength. Harza, Inc., prepared a site-specific Seismological Study for Glines Canyon Dam, dated September 4, 1989 [5]. The study resulted in an 8.3 magnitude event for the maximum credible earthquake (MCE), with a selected maximum horizontal force coefficient of 0.3 g. This design acceleration was applied to the structures during preparation of the 1992 Part 12 Report to verify their safety.

a. Arch. Bureau of Reclamation Technical Service Center personnel were able to view the upstream face of the dam, which is normally inundated, during a site visit in April 1994 (photo 15). No areas of structural distress were observable in the face at that time.

The downstream face of the arch was visually inspected by Harza Northwest, Inc., in September 1991, as part of the Part 12 inspection of the dam [4]. The face was closely examined from a cable suspended work platform. Additionally, the concrete face was tested with an impact testing hammer, resulting in relative concrete strengths ranging from 2,800 lb/in² to 3,200 lb/in². The inspection indicated that the face is in "very good" condition, with no cracking observed, and no significant seepage at joints or spalling observed. Concrete in the outlet valve operator gallery was observed to be in "excellent" condition, with no cracking or significant deficiencies observed. No deficiencies were observed in the downstream face during this examination (photo 16).

The stability of the arch section was determined to be satisfactory in a structural evaluation performed by R. W. Beck and

Associates in 1972. The arch stresses obtained were within acceptable limits. Loadings assumed were normal loading, normal plus earthquake (0.1 g), and overtopping. Finite element analyses by Harza, Inc., for the 1992 Part 12 Report indicated that the arch concrete is stable against normal and seismic loadings up to and including the MCE [5].

b. Gravity section and thrust block. No deficiencies, such as cracking, spalling, or concrete deterioration were observed on the gravity section during this examination (photo 17). In analyses for the Part 12 Report, the factors of safety for the gravity sections with the tendons installed varied from 2.10 to 2.26 for the PMF condition, which controlled over the earthquake case [3].

Factors of safety for all loading combinations on the thrust block (dead load, hydraulic load, temperature load, and earthquake load) are adequate, based on computations in 1992 [6]. During this examination, no deficiencies or evidence of structural distress was observed on the thrust block (photo 18).

c. Embankment dikes. Shallow surface settlements have been observed on both faces of the right abutment dike by previous examiners. These deficiencies do not affect the safety of this structure. During this examination, no deficiencies were observed on this dike, which contains adequate grass cover (photo 19).

The left abutment dike, constructed as a freeboard dike, has no apparent deficiencies (photo 20). This dike seldom experiences hydraulic head against it.

The structural stability of both dikes was evaluated in 1987 [7]. The minimum factor of safety for the left embankment dike is 1.24 for the upstream slope, and 1.45 for the downstream slope, during normal loading. The minimum factor of safety for the right embankment dike is 1.63 for the upstream and downstream slopes, during normal loading. Higher safety factors result from PMF loading.

No potential dam safety deficiencies exist related to the arch, gravity section, thrust block, or embankment dikes.

3. Spillway

a. Structural features. A hole in the left training wall, recommended for repair in the 1992 Part 12 Report, has been repaired.

No areas of cracking, concrete deterioration, or structural distress were reported in recent inspection reports, or observed during this examination (photos 21, 22, and 23).

Six post tensioned anchors were installed in the spillway piers in 1989, to provide structural stability during releases up to the PMF release of 63,000 ft³/s [8]. Reanalysis performed for the 1992 Part 12 Report [6] indicate adequate safety factors for all loadings, including normal, normal plus earthquake, and PMF loading.

b. Mechanical features. The gates are test operated annually and reportedly operate in a satisfactory manner. All spillway gates should continue to be maintained in a fully operable condition, and verified by periodic operating tests, until a final decision on disposition of the dam is made.

No deficiencies presently exist that would prevent the proper functioning of the spillway.

4. Powerplant

a. Structural features. The intake was inspected by robot on October 12, 1987. During this site visit, horizontal cracks and damaged areas were observed on the concrete walls in the headgate guide slot area, and downstream from the gate in the ladder well. These areas were repaired in 1988 by contract to Underwater Resources, Inc. [9] by injecting high strength epoxy into cracks, and hand placing hydraulic cement at areas of rock pockets and large voids. Most of the work was performed by divers. The tunnel was entered from the downstream end and examined internally during the 1992 Part 12 examination, and the repaired areas and other observable features were observed to be in satisfactory condition.

No deficiencies were observed in the plant building during this or previous examinations (photos 24 and 25). No conditions exist that would adversely affect operations or the safety of the dam.

b. Mechanical features. In the past, the penstocks and power tunnel (photo 26) have been internally inspected every 5 years, during the inspection for the Part 12 Report. No current deficiencies exist requiring repair. The most recent inspection occurred on August 19, 1991. Periodic repairs have occurred as needed to maintain the turbine and associated mechanical equipment.

As indicated in a 1994 assessment of interim operation of the Elwha and Glines Canyon powerplants prior to final disposition of these facilities, by the Bureau of Reclamation Pacific Northwest Region, one option for interim operation is to discontinue interior inspections of the penstock and power tunnel for the expected remaining life of the plant, due to the cost required for confined space entry requirements. This limits the amount of time that the powerplants can continue to run safely before they must be shut down, or inspected and probably repaired. If the plant is shut down, no inspections would be necessary. The need for continued interior inspections of these features for the expected remaining life of the plant should be evaluated based on the interim operation scheme for the plant.

Except for potentially affecting reservoir evacuation capability, the failure of penstocks or other features of the powerplant would not threaten the safety of the dam, although this could pose a threat to the safety of operating personnel. However, the condition of the penstocks and surge tank should be verified until a final decision on disposition of the dam is made.

5. Instrumentation. Instrumentation at Glines Canyon Dam includes measurement points on the arch dam, thrust block, gravity section, and penstock anchor, and a seepage measurement weir located downstream from the right abutment gravity section.

Vertical and horizontal measurements have yielded little change since monitoring began in 1966. Seepage monitoring at the right abutment weir has not yielded any concerns related to dam safety. Existing instrumentation indicates satisfactory performance of the dam.

G. Emergency Preparedness

1. Access to the site. The EAP lists primary and alternate routes to the dam, including access by foot, floatplane, helicopter, and boat. Additionally, organizations to be contacted to obtain alternate methods of transportation are listed in the EAP, with their telephone numbers. Other pertinent information, such as the riverflow rate ($15,000 \text{ ft}^3/\text{s}$) at which the access bridge for primary access will be overtopped, and probably fail, is discussed in the EAP.

During the preparation of this report, it was learned that the access road to the dam was partially washed out recently by runoff from heavy rains, leaving only one lane open to traffic. This roadway section is prone to continuing slumping and washouts, due

to its proximity to the Elwha River. Complete washout of the roadway is possible in the future. Since there is no other road access to the dam, access to the dam is inadequate. Adequate access is required to allow for prompt response to emergency situations at the dam. Actions necessary to ensure adequate vehicle access to the dam should be determined. It is important that all alternate access routes listed in the EAP be verified periodically, and well understood by operating personnel.

2. Communication system. Communications at Glines Canyon Dam, listed in the EAP, include commercial telephone service, the Port Angeles intra-mill telephone system, UHF radio communications (providing a direct page to PenCom, direct communications to the Daishowa America mill, communication to mobile radios, and communication with the mill paging system with interface to the telephone system), and VHF radio communications (providing communication to Elwha Dam and the Daishowa America Mill, communication to mobile radios, communication to portable radios, and communication to the Clallam County Emergency Operations Center). These communications options, and directions for accessing the various communications systems in an emergency, are listed in the EAP. The communication system is adequate.

3. Warning system. The only public warning system at the dam is a siren that sounds prior to making releases via the spillway gates. The siren was test operated during this site evaluation, and operated satisfactorily.

4. Auxiliary power. The EAP lists four sources of power for the spillway gates. The first source is the main generator (station service). Another source is a portable diesel air compressor located on the left abutment (photo 1). The auxiliary power sources are adequate.

5. Remote operation. The project is typically operated remotely from the Elwha Dam powerplant. Remote operation is available for the turbine and one gate of the spillway (the gate that the moveable hoist is connected to). Remote operation is adequate.

6. Security of the site. Security at the site consists of locked gates and doors preventing access to restricted areas of the dam and powerplant. The dam is attended daily, including weekends. No problems related to vandalism were reported by operating personnel. Security of the site is adequate.

7. Reservoir evacuation capability. Theoretically, reservoir evacuation capability is limited to elevation 503, the sill of the

power intake. At this elevation, about 40 percent of the hydraulic height, and about 75 percent of the reservoir volume, can be evacuated. This assumes that no reservoir inflow would occur during the routing period; actually, average inflow to the reservoir, as reflected in river discharge at McDonald Bridge downstream from the dam is 1,500 ft³/s, which exceeds the powerplant capacity. Therefore, in practical terms, no means of reservoir evacuation exist during most of the year, although limited evacuation would be possible during low inflow months, such as during the summer.

No records of operation exist for the low-level sluiceway. While the valve appears to be in operable condition, based on external examination, the operator is extremely reluctant to attempt to make releases from the sluiceway, due to the possibility of debris in the conduit between the inlet and the valve being passed into the conduit, and either damaging the gate leaf or becoming trapped, preventing gate closure. Additionally, design drawings show the hemispherical trashrack to be hinged, and it is possible that the trashrack could be in the open position, allowing debris to be introduced to the conduit. No signs of the trashrack were found during a recent underwater examination by Reclamation's Pacific Northwest Region, which may suggest that the trashrack is open.

It is sound engineering practice to provide for evacuation of the reservoir to low levels, for inspection or emergency drainage purposes. Although it would be preferable to have some means to drain this reservoir for emergency purposes, the operator's concerns for operating the sluiceway are understandable, and no expenditures to investigate low-level evacuation are warranted. This is even more true now that the dams are being considered for removal. Given the lack of potential dam safety deficiencies that could necessitate low level evacuation, the lack of low-level evacuation capability is acceptable in this case.

8. Operating instructions and Emergency Action Plan. An EAP for Glines Canyon and Elwha Dams was revised in 1994. Tabletop exercise tests of the EAP are conducted quarterly among operating personnel, and annually with other county, NPS, and local emergency personnel. The most recent annual test was conducted in November 1994, and consisted of a simulated onsite emergency, and the resulting communication to appropriate authorities. Based on subsequent conversations with NPS personnel, this exercise was quite useful in identifying potential problems in the communications facilities, primarily with portable communications. Deficiencies identified during this exercise will be corrected in the near future. The EAP contains adequate instructions to perform the required operations and activities in an emergency. Updating and exercising of the EAP

should continue prior to and after the anticipated acquisition of the dam by the government.

Operating personnel are well versed in procedures for operating equipment at the dam. Full instructional information is maintained at the dam. Operating instructions and Emergency Action Plan activities are adequate.

Attachments 8

Examination Party

References

Current National Park Service Dams and Related Floodplain
Inventory Report for Glines Canyon Dam (revised by examiner)

Figure 1 - Glines Canyon Dam Site Plan⁴

Figure 2 - Power Tunnel Headworks

Figure 3 - Powerplant

Figure 4 - Low Level Outlet

Photograph Pages

Review Certification

⁴All figures taken from Appendix F, The Elwha Report, Restoration of the Elwha River Ecosystem and Native Anadromous Fisheries, A Report Submitted Pursuant to Public Law 102-495, dated January 1994.

This report has been reviewed and is believed to be in accordance with the project requirements, standards of the profession, and Reclamation policy.

Reviewer: _____ Date: _____
Darian Ingram

EXAMINATION PARTY

This SEED Examination Report was prepared by:

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REFERENCES

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ELWHA RIVER
ECOSYSTEM AND FISHERIES
RESTORATION PROJECT,
Washington

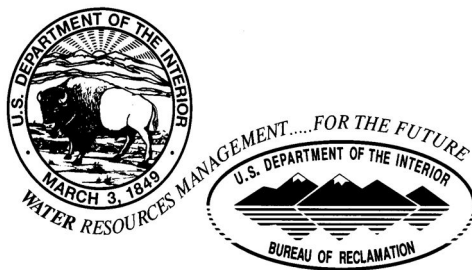
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PN-95-7

REMOVAL OF ELWHA AND
GLINES CANYON DAMS

U.S. Department of the Interior

Bureau of Reclamation
Pacific Northwest
Boise, Idaho

Region



May 1996

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ELWHA RIVER
ECOSYSTEM AND FISHERIES
RESTORATION PROJECT
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GLINES CANYON DAMS

Elwha Technical Series
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Elwha River
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Pacific
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REMOVAL OF ELWHA AND GLINES CANYON DAMS

May 1996

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ELWHA RIVER ECOSYSTEM AND FISHERIES RESTORATION PROJECT DAM REMOVAL PLAN

ABSTRACT

This technical memorandum documents the dam removal plans proposed by the Dam Removal Task Group for the purpose of establishing feasibility-level cost estimates and schedules for the removal of Elwha and Glines Canyon Dams and for subsequent restoration of the dam sites to meet project objectives.

SUMMARY

Elwha and Glines Canyon Dams are located on the Elwha River, which flows on the Olympic Peninsula of northwestern Washington State. Elwha Dam was completed in 1913 at river mile (RM) 4.9, and includes a 108-foot-high concrete gravity section, gated spillways on both abutments, and a powerhouse with four generating units rated at a combined capacity of 14.8 megawatts (MW). Glines Canyon Dam was completed in 1927 at RM 13.4 and includes a 210-foot-high concrete thin arch section, a gated spillway on the left abutment, a thrust block on the right abutment, and a powerhouse with a single generating unit rated at 13.3 MW. Elwha Dam impounds Lake Aldwell, which has a surface area of 267 acres and a storage capacity of 8,100 acre-feet at elevation of 197.0 feet. Glines Canyon Dam impounds Lake Mills, which has a surface area of 415 acres and a storage capacity of 40,500 acre-feet at elevation of 590.33 feet. Both dams are currently owned and operated by the James River Corporation.

The goal of restoring the Elwha River ecosystem and native anadromous fisheries requires removal of Elwha and Glines Canyon Dams. The plan to meet this goal includes the following objectives:

- Safely remove Elwha and Glines Canyon Dams.
- Accommodate river flows during dam removal, through diversion channels and notches.
- Facilitate sediment management through controlled releases and systematic construction schedules.
- Address environmental issues by planning work shutdowns during certain periods.
- Retain certain structures for historical preservation at Glines Canyon Dam, allowing public viewing of the site from structures on both abutments and retaining the historic powerhouse.
- Achieve reasonable costs by limiting structure removal at Glines Canyon Dam and by selecting construction methods that are economical but do not sacrifice safety.

A Dam Removal Task Group was established to prepare feasibility-level designs and cost estimates for dam removal. The design process for development of the dam removal plans consisted of the following steps:

- Review existing information, including previous studies performed by others for the *Draft Staff Report* (FERC 1993) and *The Elwha Report* (Interior and Commerce, 1994).
- Develop reasonable dam removal alternatives by considering previous designs, existing site conditions, project objectives, and current construction practices.

- Identify and address issues that could impact construction, including streamflow diversion, sediment management, safety, and environmental concerns.
- Develop an efficient construction sequence to minimize potential flood impacts and accommodate periodic shutdowns for salmon runs and potential flood flows.
- Prepare feasibility-level cost estimates for removal of each dam based on the preferred dam removal alternatives and assumed demolition methods.
- Document design studies by preparing a technical memorandum and feasibility drawings.

The reservoir behind Elwha Dam would initially be lowered by about 50 feet with the construction of a diversion channel through the existing location of the left abutment spillway. This would permit removal of the concrete gravity section and excavation of a significant portion of the upstream fill materials. The remaining reservoir would be lowered an additional 40 feet by the incremental removal of the remaining rockfill within the original river channel. All structures would either be removed or buried at the Elwha damsite, and the diversion channel would be backfilled to provide natural-looking contours on the left abutment.

Using a combination of blasting and sawcutting, the concrete arch portion of Glines Canyon Dam would be removed in 7.5-foot layers from the existing dam crest at elevation of 590.33 feet to the final streambed level at an elevation of 400 feet. Streamflow diversion would be provided by the existing power penstock for reservoir levels above 530 feet, and by a series of 15-foot-deep excavated notches alternating between the left and right sides of the concrete arch at 7.5-foot intervals, between elevations of 522.83 and 410.33 feet. The concrete thrust block, gravity wall, and embankment dike located on the right abutment of the dam would be retained for use as a public overlook. The gated spillway, penstock, gatehouse, and powerhouse structures on the left abutment would be retained for their historical significance and interpretive potential.

The total field cost of dam removal is currently estimated at \$20.2 million, including an allowance of 20 percent for contingencies. Costs for engineering and construction management would add approximately 20 percent to the overall project cost. A construction period of about 2 years is anticipated, with a recommended notice-to-proceed date of November 1. The proposed construction schedules and detailed cost estimates are included in Appendices E and F, respectively.

Additional construction requirements associated with dam removal include hazardous waste collection and disposal, removal of transmission lines, and road improvements. The U.S. Fish and Wildlife Service (the Service) has prepared a separate report on the proposed collection and disposal of hazardous waste at both dam sites, with an estimated cost to the project of \$1.2 million. Removal of 14 miles of transmission lines and poles was estimated to cost \$238,000 (plus contingencies) in 1990. Improvements necessary for the Olympic Hot Springs Road for rehabilitation following dam removal are discussed in this technical memorandum, and by the Army Corps of Engineers (Corps) in a separate report on flood protection requirements. Detailed costs of all features are discussed in a separate cost summary report in Appendix E.

CHAPTER I. INTRODUCTION

The Elwha River Ecosystem and Fisheries Restoration Act (Public Law 102-495, also called the Elwha Restoration Act), passed by Congress in 1992, requires the "...full restoration of the Elwha River ecosystem and native anadromous fisheries." Since 1911, the Elwha and Glines Canyon Dams in northwestern Washington have blocked anadromous fish passage to more than 70 miles of the Elwha River and its tributaries, limiting anadromous salmon and steelhead trout production to the lower 4.9 miles of the river below Elwha Dam. The removal of Elwha and Glines Canyon Dams was determined by the Secretary of the Interior to be the only alternative that would achieve the goal desired by Congress.

Extensive studies of potential dam removal alternatives have been performed by others; primarily Hosey & Associates Engineering Company (1989) for dam owner James River II, Inc. (now James River Corporation), and Summit Technology Consulting Engineers, Inc. (1991) for the Lower Elwha Klallam Tribe. For demonstration of the technical feasibility of removing both dams, *The Elwha Report* (Interior and Commerce, 1994) includes the work of consultants Summit Technology with R. W. Beck and Associates (1993) and provides estimated costs for the project. As part of an interagency effort, the Bureau of Reclamation (Reclamation) conducted a feasibility-level study on dam removal, sediment management, and water treatment alternatives to achieve restoration of the Elwha River. A plan of study for this work was completed in July 1994, and a Dam Removal Task Group was established to prepare feasibility-level designs and cost estimates for dam removal.

This technical memorandum discusses various dam removal alternatives, including those studied previously, and presents a preferred dam removal plan which is considered to be economical, technically feasible, and fully compatible with project objectives. This technical memorandum was utilized as a source document by the National Park Service (NPS) for the "programmatic" *Final Environmental Impact Statement I* (Final EIS, Vol. I), *Elwha River Ecosystem Restoration* (June 1995). This document will also serve as source material for the draft "implementation" Final EIS, Vol II (April 1996).

While specific methods have been developed for removal of both dams for purposes of preparing cost estimates, the Dam Removal Task Group believes there may be some merit in having contractors submit technical and cost proposals for dam removal during the contract procurement process. This concept, called "negotiated procurement," may allow for more efficient methods and for cost savings, and is often used when alternative or innovative approaches to construction are desired. This contracting process has been successfully used by Reclamation for many projects, and should result in the optimum dam removal plan for the lowest overall cost

CHAPTER II. PROJECT OBJECTIVES

The action proposed by the programmatic Final EIS is "...to fully restore the Elwha River ecosystem and native anadromous fisheries through the decommissioning of Elwha Dam and Glines Canyon Dam and removal of all structures necessary, including all or part of both dams, powerhouses, reservoirs, and associated facilities to achieve this purpose." (Interior and Commerce, 1994)

The plan of study (July 1994) identified the following objectives to achieve project restoration:

- Develop and evaluate alternative plans for removing Elwha and Glines Canyon Dams and for controlling river flows while the dams are being removed.
- Develop and evaluate alternative plans to restore the physical characteristics of the Elwha River channel and to remove and/or stabilize reservoir sediments.
- Determine the long-term equilibrium channel characteristics of the Elwha River in the area of the reservoirs and downstream to its mouth at the Strait of Juan de Fuca.
- Identify impacts to water quality and use. Develop and evaluate alternative plans for addressing water quality impacts and maintaining water supplies.
- Identify economic impacts associated with dam removal.

The Elwha River Ecosystem and Fisheries Restoration Act (P. L. 102-495) authorized acquisition of both projects at a cost of \$29.5 million, subject to the appropriation of necessary funds. The purchase is to include both dams, appurtenant works, and project lands. Upon conveyance of the projects to the United States, the Secretary of the Interior will be responsible for maintaining the dams in a safe condition until their removal. Both dams are currently listed in the National Register of Historic Places and an "Historic American Engineering Record" must be prepared to fully document each project as mitigation for their removal. Retention of portions of the original structures to the extent possible could help mitigate the impacts of dam removal and facilitate project interpretation.

CHAPTER III. EXISTING PROJECT FEATURES

A. ELWHA DAM

1. General description — Elwha Dam is located on the Elwha River, about 4.9 miles upstream from the Strait of Juan de Fuca, on Washington State's Olympic Peninsula. The dam was originally constructed in 1911-13, and impounds a reservoir (Lake Aldwell) having a surface area of 267 acres and a storage capacity of 8,100 acre-feet at an elevation of 197.0 feet. Since 1983, the reservoir level has been held within a 1-foot range of normal water surface, resulting in essentially a run-of-the-river operational mode, with flood flows required to be passed with little reduction. The primary benefit of the reservoir is power generation, although recreational benefits are also available. Maximum power generation occurs with 1,800 cubic feet per second (cfs) released equally through the four turbines, with any excess flow released from the gated spillways located on both abutments. Access to Elwha Dam is provided by an unpaved road off State Highway 112, just east of the Elwha River bridge.

Elwha Dam is currently classified by Reclamation (April 1995a) as "high hazard," a category determined by the potential for loss of life in the event of a dam failure. An inundation map for failure of Elwha Dam was prepared by Harza Northwest, Inc. (1992). Peak flows of 159,000 cfs at the dam, reducing to 109,000 cfs at the lower Elwha River, resulted from the dam-break simulation. A safety-of-dams classification code of "FAIR" is appropriate for Elwha Dam, based on current NPS guidelines. The FAIR classification indicates that no existing or potential dam safety deficiencies are recognized for normal operating conditions, that all essential elements can be expected to function under normal operating conditions, and that remotely occurring events of magnitudes approaching a probable maximum flood (PMF) or maximum credible earthquake (MCE) may cause a dam safety deficiency. Additional hydrologic and seismotectonic analyses, including the development of a new PMF and MCE using current methods, may be required in the event the dam is not removed.

Elwha Dam and its appurtenant structures were inspected for the NPS in September 1994, and an evaluation report was completed in April 1995. A copy of this report, complete with project drawings and photographs, is included in Appendix B. Detailed descriptions of each of the project features are provided below, and are based on previously published reports and available project drawings. All elevations used in this technical memorandum are based on mean sea level, and are 9.5 feet higher than the original project datum.

2. Gravity section — The concrete gravity section has a structural height of 108 feet, a crest length of 112 feet at elevation of 199.5 feet, a crest width of 8 feet, a base width of 100 feet, and an estimated volume of 12,000 cubic yards (yd³) above elevation of 90 feet. The upstream face is vertical and the downstream face is stepped, with an average slope of 0.85:1. A timber access bridge is above the dam crest and extends about 430 feet between the abutments, with chain-link fencing along both sides. Four low-level sluiceway outlets, each 5 feet wide by 10 feet high, extend through the gravity section at "invert" (lowest point of the interior) elevation of 121.5 feet, with iron slide gates provided on the upstream face.

Concrete plugs were constructed within 10 feet of the upstream end of these outlets in 1980 to permanently seal them. Currently, the downstream steps of the gravity section show signs of deterioration and spalling (the loss of surface concrete, usually caused by impact or abrasion), but the concrete mass is believed to be reasonably sound.

The concrete gravity section was originally founded on alluvial sands and gravels 80 feet thick. A structural failure (blowout) of the foundation occurred during initial filling in October 1912, resulting in loss of the reservoir beneath the concrete structure (see Appendix A, Construction History). The subsequent placement of various materials (rockfill, fir mattresses [bundles of branches], hydraulic fill, and gunite (a type of reinforced concrete) upstream of the gravity section was effective in reducing the amount of seepage through the foundation. Investigations in 1968 revealed the existence of a large void beneath the gravity section (up to 15 feet deep), and measurements in 1972 showed a seepage flow of about 70 cfs. Compressive testing of concrete cores indicated compressive strengths in the 2,000 to 2,500 pounds per square inch (lbs/in²) range. Post-tensioned, fully-grouted tendons were installed in the concrete gravity section in 1980 and 1986 to improve structural stability during overtopping and seismic events.

3. North spillway — A gated spillway is provided on the left abutment of the dam, containing four 13.75- by 19-foot Tainter (radial) gates with a maximum estimated release capacity of 18,700 cfs at elevation of 200.0 feet. The reinforced-concrete gate structure has an overflow crest at elevation of 179.2 feet, a total length of 82.3 feet at the bridge deck (elevation of 205.0 feet), and an estimated concrete volume of 2,400 yd³. A concrete, ogee-shaped overflow section is provided below gate No. 3 for fish passage. Post-tensioned, fully-grouted tendons were installed in the north spillway piers in 1980 and 1987 to improve structural stability during overtopping and seismic events. The spillway chute was excavated in bedrock and extends over 250 feet from the spillway crest to the river channel, with a width of about 76 feet. Massive concrete gravity walls are provided along both sides of the excavated spillway chute, with an estimated volume of 2,000 yd³. The steel Tainter gates have a radius of 20 feet from the trunnion pins and a total weight of 52,000 pounds. Stoplog slots are provided in the concrete piers upstream of the gates. A single gate hoist is provided for local or remote operation of the gates, with an estimated weight of 5,000 pounds. A discharge curve for all four spillway gates fully open is provided in Figure 1.

4. South spillway — A second gated spillway is provided on the right abutment of the dam, containing five 13.75- by 19-foot Tainter gates with a maximum estimated release capacity of 23,300 cfs at elevation of 200.0 feet. The south spillway was constructed in 1914, after the initial filling and structural failure of the dam foundation. The reinforced-concrete gate structure has an overflow crest at elevation of 179.2 feet, a total length of 90.9 feet at the bridge deck (elevation of 205.0 feet), and an estimated concrete volume of 1,600 yd³. Post-tensioned, fully-grouted tendons were installed in the south spillway piers in 1980 and 1986 to improve structural stability during overtopping and seismic events. The steel Tainter gates have a radius of 20 feet from the trunnion pins and a combined weight of 65,000 pounds. Stoplog slots are provided in the concrete piers upstream of the gates. A single gate hoist is provided for local operation of the gates, with an estimated weight of 5,000 pounds. A discharge curve for all five spillway gates fully open is provided in Figure 2.

Figure 1. — Elwha Dam, North Spillway Discharge Curve

new generation hard copy received 3/20/96

Figure 2. — Elwha Dam, South Spillway Discharge Curve

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5. Intake structure — A concrete intake structure is provided between the gravity section and the north spillway, with a crest length of 131.5 feet and four intake sills at elevation of 174.5 feet. The reinforced-concrete structure is founded on bedrock and has an estimated concrete volume of 3,400 yd³. The intake structure provides four upstream 12.7- by 13-foot slide gates and hydraulic hoists for the penstocks serving the downstream powerhouse, for a total weight of 40,000 pounds. Trashracks provided upstream of each penstock inlet have a combined weight of 70,000 pounds. Post-tensioned, fully-grouted tendons (9 total) were installed in the intake structure piers in 1980 and 1987 to improve structural stability during overtopping and seismic events. Concrete compressive strengths are low, ranging from 1,100 to 2,500 lbs/in².

6. Penstocks and surge tank — Four steel penstocks are provided between the intake structure and the powerhouse. Each penstock is about 190 feet long with an intake sill at elevation of 174.5 feet. The penstocks are supported on concrete saddles and within concrete thrust blocks having a total volume of about 600 yd³. The two penstocks to the north are each 9.5 feet in diameter and supply water to units 1 and 2. A 15-foot-diameter penstock to the south is supplied by two 9.5-foot-diameter penstocks at the upstream end, and branches at a surge tank to two smaller penstocks serving units 3 and 4. The surge tank is 120 feet high, with a tank diameter of 25 feet and a 20-foot-diameter riser pipe. A 30-inch-diameter penstock is provided for station service. Total release capacity of the power penstocks is 1,800 cfs while generating power. The steel penstocks have a total estimated weight of 572,000 pounds. The surge tank has an estimated weight of 275,000 pounds. A concrete walkway has an estimated volume of 50 yd³.

7. Powerhouse — The powerhouse structure is located on the left abutment downstream from the intake structure, and includes two circa-1914 horizontal double-runner turbine generators (units 1 and 2, each rated at 3,000 kilovolt amperes [kVA] and 4,800 horsepower) and two circa-1922 vertical single-runner Francis-type turbine generators (units 3 and 4; each rated at 3,330 kVA and 5,000 horsepower) for production, a smaller horizontal turbine generator for station service (at 200 kVA), and governors and other auxiliaries for the units. The maximum rated capacity of the powerplant is 14.8 MW. Penstock releases are controlled by wicket gates at each turbine generator. The control room is located in the original powerhouse and contains the main control panel, communications, and protective relays. Manually-operated butterfly guard valves are provided for units 3 and 4, in addition to the slide gates at the intake structure. The second floor of the powerhouse contains a small machine shop and transformers and circuit breakers for the units.

The powerhouse structure is of reinforced concrete and has a main floor covering about 7,800 square feet (ft²), with an estimated 6,900 yd³ of concrete and 3,800 ft² of glass-pane windows. The mechanical equipment, including the turbines, governors, and overhead cranes, has an estimated weight of 515,000 pounds. The electrical equipment, including the generators, exciters, transformers, and circuit breakers, has an estimated weight of 950,000 pounds. Discharge curves for units 3 and 4 (vertical units) with various gate openings and with the turbine runners removed (for no-load operation) are provided in Figure 3.

8. Transformer yard — A small transformer yard or substation is located on the right abutment downstream from the dam, above the access road. At the transformer yard, the generator voltage is transformed to 69 kV and conveyed about 6 miles via an overhead three-phase transmission line to the Daishowa-America paper mill (Line No. 1) or about 8 miles to the Bonneville Power Administration-City of Port Angeles substation (Line No. 2). A chain-link fence surrounds the electrical equipment at the site. The estimated weights for the electrical equipment within the transformer yard are included in the total weight provided above for the powerhouse.

9. Reservoir forebay — Between 1913 and 1919, a massive stabilizing fill of large rock and hydraulically-placed fill (sand and gravel) was placed immediately upstream from the concrete gravity section within the forebay area. An estimated 200,000 yd³ of earthen materials (including about 25 percent rockfill) must be removed to restore the original river channel. A reinforced-concrete (gunite) channel lining was constructed on the fill in the early 1920's within 200-300 feet of the dam, covering about 60,000 ft² of the forebay area between both abutments with an average thickness of 3 inches. The total estimated volume of the concrete channel lining is about 600 yd³. The channel fill includes an earthen dike extending upstream from a large concrete block at the penstock intake structure, to divide the forebay area into two distinct channels to the north and south spillways. The concrete block has a top surface at an elevation of 196 feet, and an estimated volume of 1,400 yd³.

Additional seepage control measures located within the forebay area include large fir mattresses having an estimated volume of over 2,000 yd³. A series of sheet-pile cutoffs were constructed to depths up to 50 feet below the streambed surface both upstream and downstream from the dam. A concrete-filled caisson was constructed near the toe of the dam below an elevation of 100 feet.

10. Fish screens and rearing pens — An Eicher fish-screen facility is located on the left abutment above the penstock for Unit 1, and is owned by the Electric Power Research Institute (EPRI). A small derrick boom crane was erected for installation and operation of the facility. Two large salmon rearing pens and a smaller tank-like structure were constructed on the reservoir near the right abutment. Access to the rearing pens on the reservoir is through the shop garage on the right abutment. These items are no longer used. At EPRI's direction, James River Corporation offered the material to the City of Port Angeles. Port Angeles has identified specific items that it wants to salvage; the remainder becomes part of the project.

11. Log boom — A log boom is on the reservoir and consists of logs cabled or chained together. The estimated length of the log boom is about 330 feet between abutments, and about 490 feet to the left end of the intake structure to isolate the left (north) spillway channel for controlled releases of floating logs to the river.

12. Maintenance shop and outbuildings — A maintenance shop/office and garage is located on the right abutment of the dam, with smaller, wood-frame structures located on both abutments. The maintenance shop/office has asbestos siding and floor tile, with a floor area of about 2,150 ft², including an attached garage with vehicle access doors on both ends. A garage on the left abutment has a floor area of about 630 ft².

Figure 3. — Elwha Dam Powerplant Discharge Curve (Vertical Units)

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B. GLINES CANYON DAM

1. General description — Glines Canyon Dam is located on the Elwha River within the Olympic National Park, about 13.4 miles upstream from the Strait of Juan de Fuca, on Washington's Olympic Peninsula. The dam was originally constructed in 1926-1927, and impounds a reservoir (Lake Mills) having a surface area of 415 acres and a storage capacity of 40,500 acre feet at elevation of 590.33 feet. Since 1975, Glines Canyon Dam has been operated as a run-of-the-river facility, with flood flows generally required to be released with little reduction. The primary benefit of the reservoir is power generation, although recreational benefits are also available. Maximum power generation occurs with 1,150 cfs released through the single turbine, with any excess flow released from the gated spillway located on the left abutment. Remote operation of the turbine and of one spillway gate is possible from Elwha Dam. Access to Glines Canyon Dam is provided by the Olympic Hot Springs Road off U.S. Highway 101.

Glines Canyon Dam is currently classified by Reclamation (April 1995b) as "high hazard," determined by the potential for loss of life in the event of a dam failure. An inundation map for failure of Glines Canyon Dam was prepared by Harza Northwest, Inc. in 1992. Peak flows of 650,000 cfs at the dam, reducing to 350,000 cfs at the lower Elwha River (including overtopping failure of Elwha Dam), resulted from the dam-break simulation. A safety of dams classification code of "SATISFACTORY" is appropriate for Glines Canyon Dam, based on current NPS guidelines. The SATISFACTORY classification indicates that no existing or potential dam safety deficiencies are recognized, and that all essential elements can be expected to function under all operating conditions, including such events as the PMF or MCE, which have only a remote chance of occurring. Additional hydrologic and seismotectonic analyses, including the development of a new PMF and MCE using current methods, may be required in the event the dam is not removed.

Glines Canyon Dam and its appurtenant structures were inspected for the NPS in September 1994, and an evaluation report was completed in April 1995. A copy of this report, complete with project drawings and photographs, is included in Appendix B. Detailed descriptions of each of the project features are provided below, and are based on previously published reports and available project drawings. All elevations used in this technical memorandum are based on mean sea level, and are 19.67 feet lower than the original project datum.

2. Arch dam — The varied radius, concrete, thin-arch dam has a structural height of 210 feet, with a crest length of 150 feet at elevation of 590.33 feet, a crest width of 4 feet, a base width of 46 feet, and an estimated volume of 15,200 yd³ above elevation of 400 feet. The upper 50 feet of the arch dam is reinforced near the abutments. The downstream face is vertical above elevation of 426 feet and the upstream face is curved. An abandoned low-level outlet works, or sluiceway, is located through the arch dam at centerline elevation of 450.33 feet. The outlet consists of a circular intake with a hinged, hemispherical trashrack, a 6-foot-long transition from an 11-foot-diameter to a 5-foot-diameter conduit, a 60-inch-diameter butterfly valve, and an 18-foot-long outlet transition from a 5-foot-diameter to a 6.5-foot-diameter conduit. A vertical ladder well on the downstream face of the arch dam and a horizontal adit provides access from the dam crest to the butterfly valve operator, which reportedly has not been operated since original construction. The trashrack, butterfly valve, and operator have a combined weight of about 12,200 pounds.

A concrete walkway and supporting piers were removed from the dam crest in 1969 to facilitate dam overtopping for passage of large floods up to the PMF. The dam crest serves as an overflow spillway with a discharge capacity of 86,000 cfs at the maximum water surface elevation of 611.8 feet. A concrete apron extends about 36 feet downstream from the dam between the canyon walls for erosion protection at the toe of the dam.

The downstream face of the arch dam was closely examined by Harza Northwest, Inc. in September 1991, and was found to be in very good condition, with no cracking, spalling, or significant seepage observed. Non-destructive strength testing of the downstream face resulted in relative concrete strengths ranging from 2,800 to 3,200 lbs/in². The upper 18 feet of the upstream face of the arch dam was observed during the reservoir draw-down test in April 1994 and there were no visible signs of structural distress. An underwater inspection of the arch dam was performed in 1994 by Reclamation's Pacific Northwest Regional Office using a remote-controlled camera, with no problems reported. Vertical and horizontal movements at measurement points on the arch dam, thrust block, gravity section, and penstock anchor have been satisfactory since monitoring began in 1966.

3. Thrust block and gravity section — The right end of the arch dam contacts a concrete gravity thrust block, adjoining a low concrete gravity section and a reinforced concrete wall on the right abutment. The thrust block is 20 feet wide and 30 feet long at the crest elevation of 600.33 feet, and extends up to 40 feet above the bedrock foundation. The thrust block includes an abandoned 6 feet wide log flume along the far face, with an invert at elevation of 581.3 feet. The gravity section is 55 feet long and contains a 0.7:1 downstream face batter below an elevation of 593 feet, with an 18-inch-thick wall above this elevation. The reinforced concrete wall is 55 feet long and 3 feet thick. The gravity section and concrete wall contain a 6-foot-wide, 12-inch-thick reinforced concrete deck at elevation of 600.33 feet. The thrust block, gravity section, and concrete wall have a total estimated volume of 2,100 yd³. Post-tensioned, grouted, and ungrouted tendons were installed in the gravity section in 1969, 1973, and 1989 to improve structural stability during overtopping and seismic events. Lampposts and handrails are located along the perimeter of the thrust block and reinforced concrete deck.

4. Embankment dikes — An earthfill embankment is located beyond the concrete wall and gravity section on the upper right (north) abutment, and was modified in 1969 for containment of the PMF. The embankment has a crest length of 175 feet at elevation of 612.33 feet, a crest width of 8 feet, and 1:1 slopes. Riprap slope protection is provided on the upstream slope below elevation of 595 feet, with grass cover above elevation of 595 feet and on the downstream slope. The embankment contains a concrete core wall and has a maximum height of 80 feet above the bedrock foundation. The total estimated volume of the embankment is 5,500 yd³.

A second earthfill embankment was constructed on the upper left abutment beyond the gated spillway in 1973 for containment of the PMF. The embankment has a crest length of 260 feet at elevation of 612.33 feet, a crest width of 8 feet, and 1½:1 slopes protected by grass cover. The total estimated volume of the embankment is 2,400 yd³.

5. Gated spillway — A gated spillway is located on the left abutment of the dam, with an overflow crest at elevation of 570.33 feet, and five 20- by 20-foot Tainter gates with a maximum release capacity of 63,000 cfs at reservoir elevation of 611.8 feet. Spillway releases pass through a reinforced concrete chute and flip structure to the river channel below.

The spillway structure has a total length of 120 feet at the bridge deck (elevation of 600.3 feet) and an estimated concrete volume of 2,500 yd³. Post-tensioned, fully grouted tendons were installed in the spillway piers in 1989 to improve structural stability during overtopping and seismic events. Each steel Tainter gate has a radius of 15 feet from the trunnion pin at elevation of 578.33 feet.

A single hoist motor is located on the hoist deck for gate operation. The combined weight of the Tainter gates and gate hoist is about 100,000 pounds. Stoplog slots are located in the upstream piers. A discharge curve for all spillway gates fully open is provided in Figure 4.

6. Intake structure — A 115-foot-high intake structure is located at the upstream end of the power penstock tunnel, about 200 feet from the left abutment shoreline. The lower intake chamber (at the tunnel portal) and the upper trashrack tower are reinforced concrete, with a combined volume of 450 yd³. The steel trashracks have an estimated weight of 50,000 pounds. The intake invert is at elevation of 501.4 feet. Access to the trashrack tower is by boat.

7. Power penstock tunnel — The power penstock tunnel extends 568 feet between portals through the left abutment of the dam. The upstream portion is lined with concrete on the crown and sidewalls only, with a minimum diameter of 11 feet and a length of 167 feet. A 45-foot-long, concrete-lined transition is provided for the rectangular guard gate section at station 1+92, with a sill at elevation of 503.33 feet. The downstream portion of the power penstock tunnel is concrete-lined for about 172 feet (11-foot diameter), dropping about 27 feet in elevation, and then is unlined for the final 184 feet (13.5-foot diameter). The unlined portion contains a 10-foot-diameter riveted steel pressure pipe.

8. Gatehouse and shaft — A reinforced-concrete gatehouse (floor elevation of 606.75 feet) is supported on concrete columns 17.5 feet above an 86-foot-deep, concrete-lined gate shaft extending to the power penstock tunnel invert. A 7- by 12-foot caterpillar guard gate is located within the 3- by 9-foot gate shaft for emergency closure and for penstock maintenance. Access to the power penstock tunnel is located downstream of the guard gate slot by a 2.5- by 2.5-foot access shaft. The gate and access shafts are covered by steel grating at the lower deck elevation of 589.2 feet. Access to the gatehouse is provided by a concrete footbridge. The gatehouse, support columns, and footbridge have a combined estimated concrete volume of 30 yd³. The portion of the gate shaft above the ground surface has an estimated concrete volume of 60 yd³. Gate operating equipment within the gatehouse is estimated to weigh 10,000 pounds.

9. Penstock and surge tank — A 10-foot-diameter steel penstock extends about 75 feet from the downstream tunnel portal at station 5+70 to a steel transition and surge tank. The surge tank is about 150 feet high and is located at station 6+70 (centerline). The steel tank is 20 feet in diameter, with an 11-foot-diameter riser pipe and steel support structure on concrete footings. The penstock transitions to a 11.58-foot-diameter and drops about 73 feet between station 6+90 (downstream from the surge tank) and station 7+96 (at the powerhouse). A stairway is located on the sloping penstock for access to the surge tank, downstream tunnel portal, and transformer yard. Total weight of the steel penstock pipe between stations 5+70 and 7+96 is about 168,000 pounds. The surge tank has a total estimated weight of 130,000 pounds.

10. Powerhouse — The powerhouse is located 700 feet downstream from the dam on the left abutment, and includes a vertical Francis-type turbine generator (centerline station 8+26, elevation of 403.33 feet) rated at 13,333 kVA and 17,500 horsepower for commercial use, a horizontal Pelton-type turbine generator rated at 50 kW for station service, and governors and other auxiliaries for the units. Penstock releases are controlled by wicket gates at the vertical turbine generator, with an 8.5-foot-diameter, hydraulically-operated butterfly valve provided just upstream. The unit has a maximum release capacity of 1,150 cfs while generating power. Discharge curves for various gate openings with the turbine runner removed (for greater capacity under no load conditions) are provided in Figure 5.

The powerhouse is reinforced concrete and measures approximately 40 by 60 feet, covering an area of about 2,400 ft². The superstructure (above the main floor, elevation of 410.33 feet) has an estimated concrete volume of 500 yd³ and about 1,200 ft² of glass-pane windows. The mechanical equipment, including the turbine, governors, and overhead crane, has an estimated weight of 400,000 pounds. The electrical equipment, including the generators, exciter, and transformers, has an estimated weight of 325,000 pounds. A concrete retaining wall located at the powerhouse within the tailrace area has a top width of 3 feet and is approximately 55 feet long, for an estimated volume of 140 yd³. A small metal shed near the powerhouse measures 12 by 20 feet and covers an area of 240 ft². Access to the powerhouse is provided by an unpaved road from the left (west) abutment access road (Olympic Hot Springs Road).

11. Transformer yard — A transformer yard, or substation, located on the left abutment above the powerhouse, is where the generator voltage is transformed to 69 kV and conveyed about 7 miles via an overhead three-phase transmission line (Line No. 3) to Elwha Dam. The yard contains electrical transformer equipment within a fenced area, including three single-phase transformers, a circuit breaker, and a disconnect switch. A metal shop building near the transformer yard measures 20 by 20 feet, covering an area of 400 ft². Access to the transformer yard is by unpaved road from the left abutment access road, or by stairway and footpath from the powerhouse.

12. Log boom — An extensive log boom system is located on the reservoir about 300 feet upstream from the dam, consisting of logs cabled or chained together and anchored on both abutments. Provisions have been made for boat passage, and for periodic release of accumulated debris through the log boom and over the dam crest.

13. Maintenance shop and outbuildings — Two wood-frame residences, a boat house, and a maintenance shop are located on the left abutment of the dam. The two-story residences each measure 25 by 35 feet in plan view, covering a combined area of 1,750 ft². The maintenance shop measures 27 by 50 feet, covering an area of 1,350 ft². The maintenance shop is one of the original construction camp buildings, and has the original blacksmith shop intact. The boat house measures 15 by 25 feet, with an area of 375 ft².

Figure 4. — Glines Canyon Dam, Spillway Discharge Curve

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Figure 5. — Glines Canyon Dam, Powerplant Discharge Curve

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CHAPTER IV. ENGINEERING GEOLOGY

A. ELWHA DAM

The foundation of Elwha Dam primarily consists of a variably-cemented, gravel conglomerate bedrock of the Twin Rivers Group, which directly underlies the structure. The exception is the main gravity section in the old river channel which is founded on Elwha River alluvium. Two vertical exploration holes (DH-A4 and DH-A5) were drilled in the left abutment forebay area in 1994 for core sampling and determination of bedrock depth. The conglomerate appears to be massively bedded, and moderately to slightly fractured. Three distinct joint sets were observed in the core, with generally rough and planar to irregular surfaces, which are lightly-stained-to-coated with iron oxide minerals. The first joint set dipped 5-15 degrees with a typical spacing of 2-10 feet. The second joint set dipped 20-35 degrees with a typical spacing of 1-2 feet, but as wide as 11-13 feet. The third joint set dipped 50-65 degrees with a typical spacing of 1-8 feet.

Excavations within the generally moderately to strongly cemented conglomerate are expected to produce stable, near-vertical cutslopes. Localized zones of more intense fracturing may be encountered near the blast hole locations from 1913, which may inhibit the effectiveness of blasting and result in local over-excavation. A top-of-bedrock contour map was developed for the damsite based on geologic mapping of bedrock outcrops, drill hole data, and available information from project drawings and records, and is included in *Geologic Design Data Report for Removal of Elwha Dam* (Reclamation, May 1996).

Following failure of the dam foundation and loss of the reservoir in October 1912, the forebay area immediately upstream from the dam was backfilled in stages between 1913 and 1919 to restore the reservoir and reduce seepage losses through the foundation. Heavy blasting of both canyon walls was performed in 1913 to produce material to plug an open gap beneath the main gravity section resulting from blowout of the foundation alluvium. An estimated 50,000 yd³ of rockfill was deposited into the river channel, with the boulders ranging in size from 2 to 40 tons, although considerable smaller-size material would also have been produced by the blast. An additional 6,000 yd³ of rockfill was excavated from the approach channel for the south (right) spillway and dumped into the river channel upstream from the dam prior to the blast. A thick blanket of fir mattresses (bundles of boughs) was placed on top of the rockfill within 150 feet of the dam to help reduce the seepage beneath the main gravity section, and was overlain by loose rock and earth (silt, sand, and gravel with cobbles and minor boulders) from 1913-14. Hydraulic fill was mined from glacial deposits on the right abutment high above the dam using high-pressure water jets, and transported to the forebay area through a network of flumes and pipelines from 1918-19. Laboratory testing of drill hole samples of the hydraulic fill indicates an average gradation of 66 percent sand, 24 percent gravel, and 10 percent non-plastic, silty fines.

A review of construction photographs suggests cobble-size materials, ranging up to 8 inches in diameter, may comprise 5-15 percent of the fill, and that wooden planks and timbers from the old flume system remain in the fill. In-place densities of all channel fill materials are expected to be low, possibly 70 to 80 pounds per cubic foot (lbs/ft³), with specific gravities of about 2.64. Excavations

through granular materials will likely have to be laid back to 2:1 slopes or flatter to insure stability of the fill materials in open cuts.

Bathymetric surveys of the Lake Aldwell forebay area indicated the presence of an irregular, hummocky lobe of landslide material completely blocking the Elwha River channel about 700 feet upstream from the dam, with an estimated volume of 15,000 yd³ (lobe only). The surface elevation of the slide mass ranges from elevation of 165 feet on the left abutment to elevation of 150 feet over much of the length of the lobe, with a crest length of about 125 feet and a maximum width of about 110 feet. An apparent lack of surface erosion suggests the landslide is relatively young in age and probably dates from initial filling of the reservoir or later.

The landslide materials are probably derived from the granular glacial sediments exposed upslope from the slide mass. Representative exposures of glacial sediments were estimated to contain 50 percent sand, 35 percent gravel, 10 percent non-plastic fines, and about 5 percent cobble with a trace of plus-12-inch boulders. A considerable amount of excavation may be required to achieve a stable slope within the saturated landslide materials. Reservoir draw-down rates during dam removal may have to be limited to allow time for drainage of the landslide materials, to reduce the potential for further landsliding. A reconnaissance-level review of the reservoir rim in March 1995 did not locate any other areas prone to future landsliding. Much of the reservoir shoreline consists of bedrock outcrops or compact glacial sediments which are now considered stable.

B. GLINES CANYON DAM

The foundation of Glines Canyon Dam consists of a basalt bedrock of the Crescent Formation, which directly underlies the entire arch dam section and appurtenant structures. The basalt is very fine-grained with widely-spaced fracturing.

The forebay area was examined by the Pacific Northwest Regional Office dive team using a remotely-operated vehicle (ROV). The examination failed to locate the intake trashrack for the abandoned sluiceway, which is believed to be entombed within reservoir sediments. The ROV camera showed fine sediments at the dam with numerous water-logged trees and woody debris. Bathymetric surveys of the area indicate sediment accumulation to an elevation of 445 feet.

CHAPTER V. HYDROLOGY

A. ELWHA RIVER BASIN

The Elwha River is a coarse-bed stream located on the Olympic Peninsula of northwestern Washington. The river basin is the fourth largest on the peninsula (by drainage area) and has over 100 miles of tributary streams. The river flows northward 45 miles from the base of Mount Olympus to the Strait of Juan de Fuca near the City of Port Angeles, Washington. The river falls about 4,500 feet in elevation to sea level, and has an average annual instantaneous flow of about 1,500 cfs. The watershed has a drainage area of 325 square miles; 83 percent of which is within Olympic National Park. Within the park boundaries, the watershed is pristine and water quality is excellent. The climate of the area is characterized by mild, wet winters and relatively cool, dry summers. Average annual rainfall ranges from over 100 inches at the upper end of the watershed area to about 40 inches at the mouth.

B. NORMAL STREAMFLOW

Elwha Dam is located 4.9 river miles upstream from the mouth of the river, and Glines Canyon Dam is located 8.5 river miles upstream from Elwha Dam. Streamflow has been recorded for the Elwha River at the McDonald Bridge gauging station (USGS No. 12045500), located between the two dams and representing a watershed of 269 square miles, for the periods 1898–1902 and 1920–date.

Measurements from 1898–1926 represent unregulated, or natural, flows. Since 1927, the recorded flows reflect regulation by Glines Canyon Dam. Average daily and monthly discharges, by month, recorded at the McDonald Bridge gauge are provided in table 1. Average daily discharges for each calendar day are provided in table 2. A flow distribution for the Elwha River based on daily discharge records from 1920–90 is provided in table 3. Duration tables of two mean daily flows for each month, with exceedance probabilities, are provided in table 4.

Table 1. — Mean Average Daily and Monthly Discharges, Elwha River†

Month	Daily Average	Monthly Average	
		Maximum	Minimum
October	*952	2,800	252
November	1,655	4,667	219
December	2,092	5,111	642
January	1,863	4,033	424
February	1,708	3,826	476
March	1,300	3,307	511
April	1,265	2,210	689
May	1,935	3,333	962
June	2,293	4,096	922
July	1,559	2,917	595
August	846	1,642	434
September	618	1,331	330
Source: USGS; measured at the McDonald Bridge gauging station (USGS No. 12045500)			
† Period of record Water Years 1898-1901, 1919-94; adjusted for storage since 1927.			

Table 2. — Average Daily Discharges, Elwha River

Table 3. — Flow Distribution Based on Daily Discharge Records, Elwha River (1920-90)

Flow between (cfs)	Number of Events	Percent of Total Time
0	0	0.00
250	256	0.99
500	2,351	9.13
750	3,690	14.34
1,000	3,690	14.34
1,250	3,546	13.78
1,500	2,825	10.97
1,750	2,269	8.81
2,000	1,712	6.65
2,250	1,227	4.77
2,500	929	3.61
2,750	729	2.83
3,000	524	2.04
3,250	402	1.56
3,500	307	1.19
3,750	282	1.10
4,000	164	0.64
4,250	147	0.57
4,500	114	0.44
4,750	92	0.36
5,000	72	0.28
5,250	56	0.22
5,500	50	0.19
5,750	44	0.17
6,000	25	0.10
6,250	26	0.10
6,500	25	0.10
6,750	22	0.09
7,000	16	0.06
7,250	13	0.05
7,500	15	0.06
7,750	6	0.02
8,000	7	0.03
8,250	9	0.03
8,500	4	0.02
8,750	4	0.02
9,000	10	0.04
9,250	7	0.03
9,500	4	0.02
9,750	6	0.02
10,000	6	0.02
10,250	5	0.02

10,500	5	0.02
10,750	4	0.02
Flow Between (cfs)	Number of Events	Percent of Total Time
11,000	3	0.01
11,250	6	0.02
11,500	3	0.01
11,750	4	0.02
12,000	3	0.01
12,250	3	0.01
12,500	2	0.01
12,750	0	0.00
13,000	1	0.00
13,250	0	0.00
13,500	2	0.01
13,750	0	0.00
14,000	1	0.00
14,250	3	0.01
14,500	1	0.00
14,750	0	0.00
15,000	1	0.00
15,250	0	0.00
15,500	1	0.00
15,750	1	0.00
16,000	2	0.01
16,250	0	0.00
16,500	0	0.00
16,750	0	0.00
17,000	2	0.01
17,250	0	0.00
17,500	0	0.00
17,750	0	0.00
18,000	0	0.00
18,250	1	0.00
18,500	0	0.00
18,750	1	0.00
19,000	1	0.00
19,250	1	0.00
19,500	0	0.00
19,750	1	0.00
20,000	0	0.00
20,250	0	0.00
20,500	0	0.00
20,750	0	0.00

	25,741	
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Table 4. — Duration Table of Daily Discharges by Month, Elwha River

<i>(percent of time indicated flow is equaled or exceeded)</i>				
	1,400 cfs		1,700 cfs	
Month	Percent	Days	Percent	Days
October	15.12	4.7	11.09	3.4
November	41.75	12.5	29.88	9.0
December	54.36	16.9	42.69	13.2
January	49.12	15.2	36.95	11.5
February	42.72	12.0	30.75	8.6
March	29.93	9.3	17.48	5.4
April	33.33	10.0	18.78	5.6
May	72.33	22.4	55.38	17.2
June	87.35	26.2	71.58	21.5
July	51.41	15.9	33.29	10.3
August	8.06	2.5	2.23	0.7
September	2.52	0.8	1.41	0.4

C. FLOODS

Floods in the Elwha River basin are generally the product of rain on saturated ground, and in rare cases, rain on snow-covered ground. The major floods recorded at the stream gauge have generally occurred between November and March, which coincides with the rainy season. Approximately 75 percent of the average annual rainfall occurs during these five months. The largest flood peak ever recorded on the Elwha River was 41,600 cfs on November 18, 1898, which was prior to the construction of the dams. Since dam construction, the largest flood peak recorded was 30,000 cfs on November 26, 1950. Base flow conditions reflect ground-water discharge, and melting of glaciers and residual snowpacks at the higher elevations within the basin.

Frequency floods having 15-day durations were developed for the drainage areas above Glines Canyon Dam, between Glines Canyon and Elwha Dams, and from Elwha Dam to the mouth, for recurrence intervals of 2, 5, 10, 25, 50, 100, 200, and 500 years (Reclamation, March 1995a). Seasonal flood hydrographs were also developed for the July through October period at Glines Canyon Dam, having probabilities of occurrence of 50, 20, 10, and 5 percent. The computed frequency flood peaks are provided in Tables 5 and 6. The flood hydrographs are provided in Figures 6 through 9.

Both reservoirs on the Elwha River are operated as run-of-the-river projects, with little or no effect on flood peaks. Glines Canyon Dam impounds Lake Mills, which has a surface area of 415 acres and a storage capacity of about 40,500 acre feet at elevation of 590.33 feet (Interior and Commerce, 1994). Capacity and area curves for Lake Mills are provided in Figure 10. Current sediment estimates for Lake Mills would reduce the total storage capacity by about 7,800 acre feet, nearly 20 percent. Elwha Dam impounds Lake Aldwell, which has a surface area of 267 acres and a storage capacity of about 8,100 acre feet at elevation of 197.0 feet (Interior and Commerce, 1994). Area and capacity curves for Lake Aldwell are not available. Revised area and capacity curves for both reservoirs will be developed using final design, based on latest survey data. Current sediment estimates for Lake Aldwell would reduce the total storage capacity by about 2,400 acre feet, nearly 30 percent.

Table 5. — Frequency Flood Peaks, Elwha River (in cfs)

Recurrence Interval	Above Glines Canyon Dam (255 square miles)	Between Glines and Elwha (63 square miles)	Elwha Dam to Mouth (7 square miles)
2-year	11,821	1,381	505
5-year	19,204	2,243	821
10-year	25,017	2,922	1,069
25-year	33,440	3,905	1,429
50-year	40,519	4,732	1,731
100-year	48,307	5,641	2,064
200-year	56,886	6,643	2,431
500-year	69,578	8,126	2,973

Table 6. — Seasonal Flood Peaks Above Glines Canyon Dam, Elwha River (in cfs)

Probability of Occurrence (percent)	Seasonal Flood Peak (July-October)
50	4,431
20	7,263
10	9,781
5	12,689

Figure 6. — Flood Hydrographs, Elwha River at Glines Canyon Dam

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Figure 7. — Flood Hydrographs, Glines Canyon Dam to Elwha Dam

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Figure 8. — Flood Hydrographs, Elwha Dam to Mouth of Elwha River

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Figure 9. — Seasonal Flood Hydrographs, Elwha River at Glines Canyon Dam

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Figure 10. — Capacity and Area Curves, Glines Canyon Dam

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CHAPTER VI. DISCUSSION OF DAM REMOVAL ALTERNATIVES

A. PREVIOUS STUDIES

1. FERC Draft Staff Report — The *Draft Staff Report* was issued by the Federal Energy Regulatory Commission (FERC) Office of Hydropower Licensing in March 1993. The *Draft Staff Report* was prepared in response to applications submitted by James River II, Inc. (now James River Corporation) for licenses to operate and maintain the Glines Canyon Hydroelectric Project (FERC No. 588) and the Elwha Hydroelectric Project (FERC No. 2683). The *Draft Staff Report* includes an evaluation of the potential natural resource benefits, economic costs, and environmental impacts of dam-retention and dam-removal alternatives. Dam-removal alternatives were originally prepared for the damowner by Hosey & Associates Engineering Company (1989). Staff-formulated approaches to dam removal are documented in the *Draft Staff Report*. Diversion tunnels were recommended at both damsites for the passage of flood flows up to 3,500 cfs, corresponding to the estimated 10-year, July to September flood. All concrete structures were to be demolished using drill and blast methods, with the rubble used for restoration of site topography or hauled to off-site disposal areas. Estimated costs for the diversion tunnel and structure removal amounted to \$12.3 million at Elwha Dam and \$5.5 million at Glines Canyon Dam, for a combined estimated cost of \$17.8 million (in 1990 dollars). Details of the dam removal alternatives for both damsites are described in Appendix A, Parts 3 and 4 of the *Draft Staff Report*.

2. The Elwha Report — *The Elwha Report* was prepared by agencies of the Departments of the Interior and Commerce, and by representatives of the Lower Elwha Klallam Tribe (Tribe), pursuant to Public Law 102-495 for the restoration of the Elwha River ecosystem and native anadromous fisheries. The dam removal alternatives presented in *The Elwha Report* were originally prepared by consultants to the Tribe, Summit Technology Consulting Engineers, Inc. (November 1991). These studies also assumed the removal of all existing structures from both damsites, using drill and blast methods. Streamflow diversion was to be provided by various methods, for the passage of flood flows up to 5,000 cfs, corresponding to the estimated 2-year, 7-day average flood. Both projects were to be removed concurrently, over a period of 2 to 3 years. Estimated costs for a surface diversion channel and structure removal at Elwha Dam amounted to \$8.6 million, and estimated costs for a low-level diversion and structure removal at Glines Canyon Dam amounted to \$8.4 million, for a combined estimated cost of \$17 million (in 1992 dollars). Details of the dam removal alternatives for both damsites are described in Appendix F of *The Elwha Report* (Interior and Commerce, 1994).

The Elwha Report notes that different vertical control datums were used for the original construction of Elwha and Glines Canyon Dams. Original elevations for the Elwha Project, recorded on as-built and report drawings, were reported to be 8 feet lower than the National Geodetic Vertical Datum (NGVD) of 1929. Conversely, original elevations for the Glines Canyon Project were reported to be 19.67 feet higher than the NGVD datum. Recent soundings of Lake Mills and reaches of the Elwha River used the NGVD datum. For clarity, *The Elwha Report* related all project feature elevations to the 1929 NGVD datum, referred to as mean sea level. Recent site surveys performed by Reclamation suggest the Elwha project datum is 9.5 feet lower than the NGVD datum.

B. ELWHA DAM REMOVAL ALTERNATIVES

The previous studies for the removal of Elwha Dam required the removal of all structures from the site, including the concrete gravity (main dam) section, intake structure, penstocks, surge tank, powerhouse, and both spillways. Reclamation identified minimum removal requirements at the Elwha damsite for fish passage and public safety, but the cultural significance of the site to the Tribe supported full removal of all features.

Various alternatives for dam removal have been categorized by the method of streamflow diversion used. Previous studies presented in the *Draft Staff Report*, *The Elwha Report*, and supporting documents, and recent studies performed by the Dam Removal Task Group, considered several streamflow diversion methods for removal of Elwha Dam, as discussed below.

1. Diversion Tunnel — The *Draft Staff Report* recommended construction of a 12-foot-diameter, 980-foot-long, concrete-lined tunnel through the left (north) abutment of Elwha Dam (left of the spillway), in combination with incremental notching of the main dam section, for streamflow diversion. A lake tap would be used to extend the tunnel excavation to the reservoir, and a 10- by 14-foot wheel-mounted gate would be installed within a concrete-lined shaft to provide flow control. The reservoir would be drawn down in 5-foot stages by alternating excavations at the north and south spillways to an elevation of 150 feet, after which time the reservoir would be completely drained (during the low flow period) and the main dam and channel fill materials would be removed. The diversion tunnel alternative was believed to afford the greatest degree of downstream safety during the dam-removal and sediment-control activities. This was due to its ability to control reservoir levels and safely pass high flows while minimizing the potential for uncontrolled release of water and sediment through the potentially unstable channel fill materials. Controlled-demolition and surface diversion-channel alternatives were considered to be less reliable. The estimated cost for the diversion tunnel and structure removal was \$12.3 million (1990 dollars).

The Elwha Report proposed a diversion tunnel through the right (south) abutment of Elwha Dam (right of the spillway), with an intake located upstream of the fill material behind the main dam section and an outlet about 250 feet downstream of the dam. An additional 220-foot-long pipe extension would convey the water beyond a downstream cofferdam. The diversion tunnel would be about 900 feet long and would have a finished diameter of 13 feet. The principal features would include a lake tap, gate shaft with bonneted roller-gate, concrete-lined tunnel, and energy-dissipating structure. Following construction of the diversion tunnel, reservoir levels would be lowered and streamflow would completely bypass the demolition area. The lake tap inlet would be lowered in 10-foot stages by controlled blasting as the reservoir is drawn down. The tunnel and shaft would be plugged following restoration of the river to its natural channel. The estimated cost for the diversion tunnel and structure removal was \$14.5 million (1992 dollars).

The Dam Removal Task Group believes significant risk and expense would be associated with the construction of a temporary diversion tunnel at Elwha Dam. Excavation and disturbance of the bedrock was also identified as a concern by the Tribe, due to its cultural significance. Since other streamflow-diversion methods would be technically feasible at a lower cost, the diversion tunnel alternative was eliminated from further consideration.

2. Surface Diversion Channel — *The Elwha Report* also proposed the excavation of a surface diversion channel on the left (north) abutment to permit streamflow to bypass the dam structures, with reservoir levels controlled by diversion-channel elevations. The reservoir would first be lowered using the existing power penstocks and south spillway (on the right abutment). A cofferdam would be constructed upstream of the north spillway (on the left abutment) to permit removal of the spillway structure and excavation of a diversion channel through bedrock to elevation of 135 feet (20 feet lower than proposed in the *Draft Staff Report*). The cofferdam would be constructed of reinforced rockfill for overtopping protection, with an impermeable geomembrane located on the upstream face. The reservoir would be lowered further by staged excavation of the upstream cofferdam and streamflow diversion through the excavated channel, to permit removal of the remaining structures. Following removal of the dam and restoration of the natural river channel, the diversion channel would be backfilled and revegetated. The estimated cost for the surface diversion channel and structure removal was \$8.6 million (1992 dollars).

The Dam Removal Task Group selected this basic alternative for further development, based on anticipated costs and likelihood of success. Although higher channel elevations were considered, requiring less rock excavation and larger cofferdams, the analyses favored the deeper rock-cut channel for streamflow diversion, allowing more of the channel fill excavation to be performed under dry conditions.

3. Other Alternatives — A significant concern for the removal of Elwha Dam is the potential instability of the channel fill materials placed to cutoff seepage beneath the dam during its early history (1912-19; see Appendix A), which if disturbed, could produce a piping failure of the dam foundation similar to that which occurred in 1912. Several options which address this concern were evaluated by the Dam Removal Task Group and are discussed below.

a. River Channel Realignment — This option would provide a new river channel through the bedrock on the left (north) abutment, along the existing alignment of the north spillway. The new channel would be excavated as necessary to meet fish-passage criteria. The technical advantage of this option is that it avoids disturbing the existing dam and upstream channel fill blanket materials, and thus the potential for piping failure is no greater than under the existing conditions (i.e., minimal). However, this alternative was determined to be unacceptable due to its failure to restore the natural river channel and significant cultural sites within the dam and forebay area, and its excessive excavation and disturbance of bedrock (also of cultural significance).

b. Upstream Cutoff at Dam — Another option consists of driving large diameter (5-6 feet) piles through the channel fill at the upstream face of the dam and into the bedrock, to reduce the seepage flow and blowout potential during excavation of the channel fill. Although the piles would be driven as close together as possible, a positive seal between the piles would not be provided. Advancing the piles through the rockfill, boulders, and logs in the channel fill could be very difficult, and its effectiveness in controlling seepage beneath the dam could be minimal.

Professor Arthur Casagrande described the effects of imperfect cutoffs and provided equations for calculating cutoff efficiency (Casagrande, 1961). Assuming the depth of channel fill to be cutoff is 70 feet, the base width of the dam is 97 feet, eleven 6-foot-diameter piles are required to cross the channel at the upstream face of the dam, and an average ¼-inch gap remains between the individual piles, the cutoff efficiency would only be 28 percent. That is, 72 percent of the seepage flow occurring without the cutoff piles would still pass through the gaps in the pile wall. If the number of piles required and/or the average width of the gaps increased, the cutoff efficiency would drop to even lower values. Thus, with the potential difficulties in installing the piles, and the questionable effectiveness of the resulting cutoff, this option would not appear to be feasible.

c. Foundation Stabilization at Dam — A third option includes backfilling the void beneath the dam with cement grout or tremie¹ concrete to stabilize the dam foundation. This option may be difficult to achieve due to the large seepage flow in the channel fill. A material with a controlled set-time (such as a chemical grout) or admixtures would be needed to accelerate the setting time of cement grout or concrete. Shannon and Wilson, Inc. (1968) drilled four holes through the dam in 1967. Three of these holes penetrated the upstream face of the dam at different elevations. Based on the water levels measured in these holes, it was concluded that the primary water barrier is the concrete channel lining (gunite), hydraulic fill, and fir mattresses above elevation of 148 feet, and that the underlying rockfill is essentially free-draining. Shannon and Wilson recommended that no attempt be made to backfill or grout the void space beneath the dam, because this would significantly reduce the existing drainage, change the flow paths, and possibly initiate piping of the underlying sands and gravels. This appears to be a sound interpretation of the data that was collected, and any such treatment would have to strengthen the existing materials below and downstream of the void, as well as fill the void itself.

d. Downstream Berm at Dam — Construction of a rockfill berm against the toe of the dam was considered to stabilize the foundation of the structure so that the upstream fill could be breached, and the dam notched down, in a controlled fashion. When the dam was reconstructed in 1913, the downstream area was first filled in and a stable-though-pervious condition was attained with the reservoir at an elevation of 118 feet. It appears that the upper 20 feet of the downstream fill has been eroded since the reconstruction due to spillway discharges. If this free-draining fill were to be replaced and strengthened to provide an appropriate safety factor for the higher water level that would exist when the fir mattresses were breached, a stable condition would result. The upstream fill would be slowly breached to monitor the piezometric levels within the dam and the downstream flows. If the piezometric levels and downstream flows increased, it would be necessary to wait until the reservoir recedes and the flows reduce before proceeding with the excavation. Several problems were identified for this option, including:

- the berm would have to resist streamflows and possibly flood flows during notching of the dam,
- a source of rockfill would be needed for the berm construction, and
- removal of the berm materials would be required following demolition of the dam.

¹ A "tremie" is the pipe used to convey the fresh concrete from the source, usually a concrete truck, to the intended placement location. The term is used generically to describe the process of placing concrete underwater.

e. Restore Sluiceways for Low-level Outlets — The existing concrete gravity section includes four 5- by 10-foot sluiceway openings with an invert at elevation of 121.5 feet. Although these openings were plugged with concrete within 10 feet of the original iron gates on the upstream face, the plugs could be removed and the low-level release capacity restored with the necessary excavation of the upstream channel fill materials. This upstream excavation within potentially unstable materials could result in a requirement for the passage of flows down a steep-cut slope, unless the iron gates could be restored or replaced to control streamflows. A downstream stability berm would be required to prevent blowout of the foundation materials under the higher reservoir heads. This option presented several technical concerns with no apparent cost savings.

C. GLINES CANYON DAM REMOVAL ALTERNATIVES

The previous studies for the removal of Glines Canyon Dam required the removal of all structures from the site, including the arch dam, thrust block, gravity section, embankment dikes, gated spillway, intake structure, gatehouse, penstock, surge tank, and powerhouse. For the current studies, Reclamation identified minimum removal requirements at the site for fish passage and public safety. The thrust block, gated spillway, and powerhouse were proposed to be retained to minimize the costs of dam removal and provide opportunities for cultural and historical interpretation of the site. Since the dam is currently listed in the National Register of Historic Places, this would help minimize the loss of historic resources.

Various alternatives for dam removal have been categorized by the method of streamflow diversion used. Previous studies presented in the *Draft Staff Report*, *The Elwha Report*, and recent studies performed by the Dam Removal Task Group, considered several streamflow diversion methods for removal of Glines Canyon Dam, as discussed below.

1. Diversion tunnel — The *Draft Staff Report* recommended the construction of a 12-foot-diameter, 925-foot-long, concrete-lined tunnel through the left abutment of Glines Canyon Dam, in combination with incremental notching of the main dam section, for streamflow diversion. A lake tap would be used to extend the tunnel excavation to the reservoir, and a 9- by 12-foot wheel-mounted gate would be installed within a concrete-lined shaft to provide flow control. The reservoir would be drawn down in 5-foot stages by alternating excavations between the left and right sides of the concrete arch, with the concrete blasted into the river channel below. Floods in excess of the capacity of the notches and diversion tunnel would overtop the remaining crest of the dam. The additional release capacity of a low-level diversion tunnel was recommended for a conservative estimate without further investigations. The report indicated the most economically desirable option could be simply to lower the dam incrementally using the existing bypass capacity. Other diversion options studied included a surface diversion channel to the existing intake structure, and the construction of a low-level outlet through the arch dam. The majority of the demolition work was to be performed during the low-flow period from July through September. The estimated cost for the diversion tunnel and structure removal was \$5.5 million (1990 dollars).

The Elwha Report proposed a diversion tunnel through the left (west) abutment of Glines Canyon Dam, with an intake near the existing power tunnel intake and an outlet about 100 feet downstream of the powerplant. The diversion tunnel would be about 800 feet long and would have a finished diameter of 13 feet. The principal features would include a lake tap, gate shaft with bonneted roller-gate, concrete-lined tunnel, and energy-dissipating structure. Following construction of the diversion tunnel, reservoir levels would be lowered and streamflow would completely bypass the demolition area. The lake tap inlet would be lowered in 10-foot stages by controlled blasting as the reservoir is drawn down. A cofferdam constructed just upstream of the tunnel outlet would permit construction of a haul road to the base of the dam for removal of concrete rubble. The diversion tunnel and gate shaft would be plugged following removal of the dam. The estimated cost for the diversion tunnel and structure removal was \$12.4 million (1992 dollars).

The Dam Removal Task Group believes significant risk and expense would be associated with the construction of a temporary diversion tunnel at Glines Canyon Dam. Since other streamflow diversion methods would be technically feasible at a lower cost, the diversion tunnel alternative was eliminated from further consideration.

2. Low-level Diversion — *The Elwha Report* includes an alternative to enlarge the existing 5-foot-diameter sluiceway outlet in the arch dam section. Construction of an enlarged hole through the arch dam would alter stress patterns in the concrete structure. A finite element analysis of the structure by R. W. Beck and Associates in *The Elwha Report* indicated a 10-foot-diameter conduit (13-foot-diameter excavation) constructed through the arch dam would not overstress the concrete or cause a rupture of the structure. Construction of the new outlet would require the underwater removal of the existing trashrack and lake sediments, and the installation of a large steel bulkhead on the upstream face of the dam. Reservoir releases would continue to be made through the power penstock until completion of the new outlet. Reservoir releases through the new outlet would be controlled to an elevation of 460 feet by a 117-inch-diameter fixed-cone valve. Below elevation of 460 feet, streamflow would pass through gated notches in the remaining portion of the dam in a progressive stepped approach, as described below. The estimated cost for the low-level outlet and structure removal was \$8.4 million (1992 dollars).

The *Draft Staff Report* briefly described a similar diversion alternative, requiring a new 7.5-foot-diameter outlet through the arch section with a 6-foot-diameter regulating valve for low-level releases, but structural analyses were not performed to confirm feasibility.

Although the Dam Removal Task Group felt this alternative would be technically feasible, the element of risk associated with underwater construction, and the probable interference of debris and sediments at the upstream face of the dam on the bulkhead installation and operation of the outlet, would be of concern. There did not seem to be a clear advantage in construction costs for this alternative, and design costs would be higher for the necessary finite element analyses.

3. Notches Through Dam — A third alternative proposed in *The Elwha Report* would allow streamflow to pass through gated notches in the dam structure at various elevations, allowing reservoir levels to be progressively lowered in stages. After lowering the reservoir using the spillway and power penstock, the portion of the structure above the reservoir level would be demolished and removed, using either a barge or a boom crane. This concept assumes the power penstock would first be modified at the powerplant by the installation of an 11-foot-diameter fixed-cone valve for reservoir releases above elevation of 515 feet. Construction of nine gated notches at 12.5-foot intervals between sill elevations 490 and 390 feet (streambed) was assumed to require the underwater installation of a large steel bulkhead at each notch location. An 18- by 25-foot slide gate would be installed at the upstream end of each notch to facilitate removal of the bulkhead and to regulate releases for sediment management. A stability analysis of the arch dam was performed to demonstrate safe passage of a 25,000 cfs flow through a 17-foot-wide by 25-foot-deep notch with 15 feet of surcharge over the dam crest. The estimated cost for the structure removal utilizing gated notches was \$9.1 million (1992 dollars).

The recommended alternative in the *Draft Staff Report* included incremental notching of the dam in combination with the construction of a diversion tunnel. The notches would each cover one-half the crest length of the dam, alternating from side to side at 10-foot intervals to the riverbed.

The Dam Removal Task Group selected this basic alternative for further development, based on anticipated costs and likelihood of success. Elimination of the proposed powerplant modifications for installation of an 11-foot-diameter fixed-cone valve would lower the estimated cost in *The Elwha Report* to about \$7.6 million. Penstock releases could be controlled by the existing wicket gates, with the turbine runner removed to ensure operating capability under no-load conditions. A non-gated notch alternative which could be constructed without an upstream bulkhead could produce further cost savings, provided the objectives of the sediment management plan could still be achieved.

4. Other Alternatives — The *Draft Staff Report* and *The Elwha Report* briefly discussed a channel bypass alternative which would use the existing power intake and penstock for streamflow diversion. A small diversion embankment would divert flows at the upper end of the reservoir into a trapezoidal channel running along the west edge of the reservoir for 8,000 to 9,000 feet. The *Draft Staff Report* indicated the channel would have a 23-foot bottom width and 1:1 sideslopes, with a capacity of 3,500 cfs. Numerous potential problems and very high construction costs were identified for this alternative, and no further studies were performed.

The Dam Removal Task Group looked at potential variations of the basic dam removal alternatives for Glines Canyon Dam documented in the earlier reports. Construction sequence, structure removal requirements, concrete demolition methods, and waste disposal opportunities were all investigated.

Previous studies assumed a construction sequence for which both dams would be removed at the same time, to shorten the overall construction schedule. However, if the removal of Glines Canyon Dam did not begin until after the removal of Elwha Dam, a flood surcharge space could be provided in the upstream reservoir (Lake Mills) to provide a flood control benefit for the Elwha Dam operations. Downstream releases could then be regulated using the spillway gates and power penstock at Glines Canyon Dam, to reduce the river control costs for removal of Elwha Dam. The relatively small storage capacity of the downstream reservoir (Lake Aldwell) would reduce its potential value as a sediment trap for removal of Glines Canyon Dam first, and anadromous fish passage upstream from Freshwater Bay is not improved until Elwha Dam is removed. The recommended dam removal plan retains Glines Canyon Dam until the completion of the surface diversion channel at Elwha Dam, for flood control benefits during the early excavation operations. Dam removal at both sites would be completed by November of the second year.

Minimum structure removal requirements for river restoration at Glines Canyon Dam include the removal of the majority of the arch dam section between the rock abutments and above the streambed. Portions of the arch dam abutments and base could possibly remain to the extent they are shown to be stable and outside minimum channel limits to accommodate flood flows and fish passage.

To ensure public safety, the log boom, trashrack tower, and surge tank tower should be removed, and the openings for the power penstock tunnel and gate shaft should be sealed. In addition, minimum removal requirements include the removal of any hazardous materials and much of the electrical equipment from the powerplant area, including transmission poles and overhead wires. The left abutment spillway and the right abutment thrust block, gravity section, and earthfill embankment may be left intact to serve as a public overlook for interpretation of the damsite, and the powerplant structure may be modified to serve as an interpretive center. The existing wood-frame structures on the left abutment may be found to be useful during and following construction for various purposes.

Previous studies assumed concrete demolition would be performed using conventional drill-and-blast techniques. A variety of alternative demolition methods were evaluated for the current studies, including diamond-wire sawcutting, stitch drilling, chemical splitting, hoe-ramming, high-pressure water-blasting, flame-cutting, and combinations of these methods (see Appendix F). The diamond-wire sawcutting and hoe-ramming methods of concrete demolition seemed to provide some potential advantages, warranting further study. Diamond-wire sawcutting was extensively used to breach Waddell Dam, a multiple-arch concrete dam in Arizona, under a negotiated contract for Reclamation. Bluebird Dam, a 58-foot-high concrete arch dam located within the Rocky Mountain National Park in Colorado, was completely demolished using a boom-mounted hydraulic impact hammer (“hoe-ram”).

The size and shape of concrete waste materials, and the associated handling and disposal requirements, are dependent upon the material composition (strength and presence of reinforcing steel) and demolition method selected. While concrete rubble would result from blasting and hoe-ramming, sawcutting would produce smooth-faced blocks of large sizes. Previous studies assumed concrete rubble would be removed using a barge on the reservoir, or trucks on a downstream haul road within the river channel. A downstream haul would only be feasible when streamflow diversion is provided by the power penstock or by a diversion tunnel, and would be located between the existing powerplant and the base of the arch dam section. The removal of Bluebird Dam used air-lifting extensively, while large cranes or cableways could serve the narrow Glines Canyon site effectively. Conveyor belts would possibly be feasible for shorter haul distances and for smaller materials. Waste materials would be disposed of in either land or ocean sites, or commercial uses could be found for some of the materials. Steel items could probably be salvaged for recycling, and possibly some of the equipment could be salvaged for its historical or commercial value.

CHAPTER VII. PROPOSED PLANS FOR DAM REMOVAL

The Dam Removal Task Group developed the following dam removal plans for the feasibility-level designs and cost estimates. These plans are considered to be technically feasible, economical, and compatible with the sediment management plans and project objectives. Negotiated procurement is recommended for future contracting of the demolition work, by which technical proposals would be solicited for selection based on both technical merit and cost (see Chapter XI). This contracting process has been successfully used by Reclamation for many projects, and should result in the optimum dam removal plan.

A. ELWHA DAM

1. Decommissioning of powerplant — This would include any work at the Elwha Dam powerplant required in advance of the dam removal contract work, including de-energizing the electrical equipment, preparing selected equipment for long-term storage, removing hazardous materials, re-routing transmission lines, replacing the grounding grid for the electric power utility, and removing smaller items of historical value. The turbine runners would be removed from the two vertical units (3 and 4) to permit no-load releases through the 15-foot-diameter penstock. This work would be performed under contract. Hazardous waste collection and disposal is covered in Chapter VIII, Section A.

2. Site access and mobilization — The right (south) abutment of Elwha Dam can be accessed by an existing, unpaved road off of State Highway 112, located just east of a bridge over the Elwha River. Site access improvements may be required for approximately 0.4 miles of permanent road, for widening as necessary to accommodate construction traffic, and for normal maintenance of the roadway surface and drainage culverts during the contract period. No other improvements to the existing road are anticipated.

Access to the left abutment of Elwha Dam is currently provided by an existing single-lane timber bridge across the dam crest from the right abutment. This access will disappear soon after the dam removal work begins. Alternate access may be provided by an old horse trail/logging road crossing the left abutment ridge from Dan Kelly Road off of State Highway 112, across public (Washington State Department of Natural Resources) and private land. Significant improvements to the roadway width and grade would be required for a distance of over 1.2 miles for this trail to be used for equipment travel, including the construction of at least one culvert crossing to reroute a small creek. Although the cost estimate for dam removal assumes the use of a floating bridge (“flexifloat”) for construction access to the left abutment for a period of time before the reservoir is drained (currently estimated at 75 days), the alternate left abutment access road should be investigated further during final design.

Temporary haul roads will be required to be established within the forebay area for excavation of the channel fill materials, with a small allowance included in the cost estimate. Maximum grades should generally not exceed 12 percent.

Mobilization costs are estimated to represent about 5 percent of the cost of dam removal, and would include costs for a dragline and two large mobile cranes with 200-foot booms. Crane pads will be required on the intake structure for removal of the left (north) spillway gates and the penstocks, on the right abutment for removal of the south spillway gates, and near the powerhouse for removal of the mechanical and electrical equipment. Construction power is assumed to be supplied by a single 150 kVA engine-generator, although alternate sources of electric power may be available. The cost of supplying electric power from the local utility or from the Bonneville Power Administration should be investigated during final design.

The current estimate assumes no constraints on construction activities will be necessary for protection of threatened and endangered species. Potential nesting sites for the spotted owl and marbled murrelet (both threatened and endangered species) exist near the State Highway 112 bridge and in the upper reservoir area. Observation of these nesting areas by U.S. Fish and Wildlife Service occurred during the 1995 nesting season (May 1-August 15) and is planned for the same period in 1996. If these nests are found to be active, “permits to take” (issued by the Service) should be investigated in lieu of placing noise limits and other constraints on the construction activities. Estimates of blasting parameters for removal of Elwha Dam were developed by Reclamation for use in modeling environmental noise levels during construction. The results of this model are reported in Appendix G, “Estimated Blasting Parameters for Noise Analysis — Elwha and Glines Canyon Dams.”

3. Streamflow diversion — A surface diversion channel will be excavated through the left (north) spillway channel to lower the reservoir level for removal of the concrete gravity section from the original river channel and much of the fill material from the forebay area. Details of the proposed diversion plan are provided in the following paragraphs. The discharge capacity of the proposed surface diversion channel is shown in Figure 11. Average daily discharges are assumed for purposes of estimating reservoir levels and diversion requirements.

The contractor is assumed to begin site mobilization in November, continuing through the winter months. Glines Canyon Dam would be operated for flood control to the maximum possible extent during this period, passing normal inflow through the power penstock while using available reservoir storage in Lake Mills to reduce flood peaks. Lake Aldwell would be drawn down by March 1 to an elevation of 182 feet, using the right (south) spillway and the 15-foot-diameter penstock for reservoir releases. A temporary cofferdam (about 8-10 feet high) would be constructed within the left (north) spillway approach channel about 300 feet upstream from the dam for unwatering of the channel area to permit removal of the concrete lining (gunite cap). A double-wall sheet-pile cofferdam (about 20 feet high) would be constructed across the approach channel from the left abutment to the existing concrete block (within 100 feet of the north spillway), for construction access to a crane pad on the intake structure and for flood protection. The sheet-piles would be driven to bedrock to provide a positive cutoff. The north spillway gate structure would be removed, and a diversion channel would be excavated within bedrock to an elevation of 135 feet. A dragline would be used within the reservoir upstream of the sheet-pile cofferdam for excavation of the temporary cofferdam and channel fill materials (sands and gravels), as necessary, to establish a deeper approach channel.

Following the spring runoff (May and June), the reservoir would again be lowered to an elevation of 182 feet. The construction of a smaller gravity wall (about 8 to 10 feet high) immediately downstream of the sheet-pile cofferdam would permit the removal of the sheet-pile cofferdam under a balanced reservoir head. The cofferdam foundation would remain as a bedrock plug within the diversion channel. The right half of the gravity wall would be breached during the low-flow period (beginning in August), lowering the reservoir to an elevation of 174 feet (below the power penstock

intake sill). The gravity wall could be constructed from precast concrete blocks (Doublewal™ units) used previously for construction of the temporary cofferdam.

Using the left half of the gravity wall as a working platform, a portion of the bedrock plug would be excavated (by blasting) to a maximum depth of 15 feet below the water surface, releasing an instantaneous peak flow of up to 3,000 cfs and lowering the reservoir to an elevation of 167 feet within a matter of hours. Subsequent drilling and blasting of the channel plug in stages from the bedrock surface would lower the reservoir to elevations of 160, 152, 144, and 139 feet, within a 2-3 week period.

Streamflow diversion would continue through the newly excavated channel for one full year while the channel fill materials are removed. Reservoir levels for average streamflow conditions will vary between elevations of 138 and 144 feet. The excavated surface of the concrete gravity section would remain above the level of the channel fill for flood protection, until the channel fill becomes predominantly rockfill and erosion-resistant. After the channel fill materials have been excavated to about an elevation of 145 feet, the remaining portion of the concrete gravity section would be removed to an elevation of about 90 feet. During the low-flow period of the second year, excavation within the channel fill materials would progress below the water level within the diversion channel, resulting in the diversion of streamflow along its original course through the damsite. The river channel would be lowered in 2-foot increments along its full length until reaching the desired grade at an elevation of 95 feet. This work would be performed over a 3-month period from August through October.

4. Structure removal — All concrete structures will be either removed or buried at Elwha Dam, and the left (north) spillway outlet channel will be backfilled to restore the site to approximate pre-dam conditions. Wood-frame structures will be retained for use during the demolition contract, then removed. All mechanical and electrical items, timber, glass, fencing, and hazardous materials will be removed from the site for disposal. The structure removal plan for Elwha Dam consists of four stages described below, and as depicted on drawings 1440-D-150 through -153 (Appendix C).

Stage One — Surface Diversion Channel Development (see Drawing 1440-D-150)

a. With the reservoir at its normal level, provide construction access to the left abutment using a floating bridge (“flexifloat”) from the right abutment. Alternative access may be provided by a barge from the right abutment or from the Indian Creek boat ramp (located at the upper end of the reservoir), or by construction of a temporary access road from State Highway 112 along an existing trail (about 1.2 miles) as described in Section 2 above. Deliver a mobile crane, bulldozer, and other equipment as needed to the left abutment. Remove the log boom as required.

b. Draw the reservoir down to the lowest possible level using both spillways and the 15-foot-diameter penstock (units 3 and 4). (Glines Canyon Dam will be operated for flood control during this period to the maximum possible extent.)

c. Establish a temporary stockpile area on the left abutment with a storage capacity of about 3,400 yd³. Construct an access road to the left (north) spillway approach channel area (maximum 12 percent grade).

d. Construct an 8- to 10-foot-high temporary cofferdam at the upstream end of the left (north) spillway approach channel as required to unwater the channel area. Twenty-four precast

concrete Doublewal™ units (or a similar product), backfilled with pervious sand and gravel were assumed for the cost estimate. Each precast unit is 4 feet high, 8 feet long, and 8 feet wide, with a weight of about 5 tons. Remove the exposed concrete channel lining (gunite cap) between the temporary cofferdam and the north spillway, and place the concrete rubble (about 300 yd³) in the temporary stockpile area. Maintain diversion releases through the south (right) spillway and 15-foot-diameter penstock.

e. With the temporary cofferdam in place, construct from the left abutment a 20-foot-wide, double-wall sheet-pile cofferdam across the north spillway approach channel to the existing concrete block (elevation of 196 feet). Drive sheet piles through the existing fill (surface elevation of 175 feet) to refusal in bedrock (approximate elevation of 170 feet). Using tiebacks for structural support, backfill the sheet-pile cofferdam with clean sand and/or gravel from a local commercial source (assumed necessary to avoid the processing of available site materials). The total weight of sheet-piles and tiebacks assumed for the cost estimate is 200,000 pounds, with 2,000 yd³ of pervious backfill required. Drill and blast any bedrock within the surface diversion channel limits upstream from the sheet-pile cofferdam foundation to an elevation of 135 feet, but leave the shotrock in place.

f. Permanently close the slide gates for the two 9.5-foot-diameter power penstocks. Construct a 40-foot-wide crane pad between the existing concrete block and the intake structure, using additional sheet piles and pervious backfill. Remove the temporary cofferdam from the approach channel before the reservoir rises, and stockpile the precast concrete units and backfill materials for later use.

g. Move the mobile crane with 200-foot boom from the left abutment, across the sheet-pile cofferdam, and onto the crane pad. Remove the 9.5-foot-diameter penstock pipe sections using cutting torches, a bulldozer, and the crane. Deliver the penstock pipe sections to the existing access bridge on the dam crest (or to an alternate location) for removal by a second mobile crane and subsequent delivery to a flatbed truck on the main access road.

h. Remove the four spillway gates and the gate hoist using the mobile cranes (as was done for the penstock pipe sections). Demolish the left (north) spillway gate structure and excavate the downstream portion of the diversion channel to invert elevation of 135 feet. Drill and blast the reinforced concrete structure in 1 or 2 shots, then blast the bedrock in 3 or 4 layers. Move an estimated 2,900 yd³ of concrete rubble and 7,600 yd³ of rock fill to the adjoining penstock site. Pile the waste materials outside the right wall of the left (north) spillway and upstream from the concrete wall at the powerhouse to a stable configuration, using a bulldozer. The stockpile area on the left abutment may also be used for temporary waste disposal. Concrete gravity walls downstream from the gate structure and outside the limits of the diversion channel may be left in place.

Stage Two — Streamflow Diversion and Powerhouse Demolition (see Drawing 1440-D-151)

a. Construct a temporary gravity wall at the toe of the sheet-pile cofferdam (between the cofferdam and the cutslope for the diversion channel) to the reservoir surface elevation. Thirty-four precast concrete Doublewal™ units (or a similar product) backfilled with pervious sand and gravel (as used for the upstream cofferdam) were assumed for the cost estimate.

b. Equalize the reservoir head on both sides of the sheet-pile cofferdam by filling the interspace between the cofferdam and the gravity wall with water. Breach the sheet-pile cofferdam by removing the sheet piles using a crane (starting at either end). Remove the sheet piles and backfill materials from the site. Leave the bedrock channel plug in place.

c. Breach the temporary gravity wall by removing 23 precast concrete units from the right half of the structure, and place 22 of the precast units at the upstream face of the left half of the structure (on the bedrock channel plug). This will provide a large working platform above the left half of the bedrock channel plug for operation of drilling equipment. Streamflow will begin to pass through the diversion channel above the bedrock surface (elevation of 170 feet), and possibly through the 15-foot-diameter penstock (depending upon the inflow rate).

d. Excavate an approach channel within the reservoir to an elevation of 135 feet, from the landslide area (approximately 800 feet upstream from the dam) to the bedrock channel plug. A bottom width of about 40 feet and 2:1 slopes in the existing hydraulic fill materials were assumed, representing an in-place volume of about 60,000 yd³. The northern channel slope will be formed by the existing bedrock face. The required excavation of the channel fill materials (sands and gravels) may be performed from a floating bridge or barge, using a dragline or clamshell. Excavated materials will either be hauled directly off-site to an approved disposal area, or placed temporarily within the right spillway approach channel for later removal. The cost estimate assumes the excavated materials are placed in the approach channel area for later removal.

e. With the upstream and downstream channels excavated to an elevation of 135 feet, begin controlled blasting of the bedrock channel plug in five stages from elevation of 170 feet to elevation of 135 feet. The first stage will be drilled from the working platform area, and will include removal of the precast concrete units and a portion of the bedrock to an elevation of 160 feet. A trapezoidal channel will be created with a bottom width of about 15 feet, and will lower the reservoir surface below an elevation of 170 feet. The second, third, and fourth stages will be drilled from the bedrock surface at an elevation of 170 feet, to elevations of 152, 144, and 135 feet, respectively. The fifth stage will remove the benches remaining from the previous stages to produce the final channel width of 30 feet, allowing the reservoir water levels to stabilize between each stage at increasingly lower elevations. Following each blast, excavated rock will be washed downstream, to be removed from the diversion channel and dumped onto the penstock site using the crane and a clamshell bucket. The total in-place volume of the bedrock within the diversion channel plug is about 2,400 yd³. (The removal of Glines Canyon Dam can begin following the successful completion of the diversion channel at Elwha Dam.) Sediment releases will be initiated from Lake Aldwell during this period.

f. Following lowering of the reservoir below the intake sill elevation, close the remaining slide gates and remove the 15-foot-diameter penstock using cutting torches, a bulldozer, and the crane.

g. Construct a temporary ramp fill from the existing concrete block (elevation of 196 feet) to the existing right spillway channel (elevation of 178 feet) at a maximum grade of 15 percent, using materials excavated for the upstream approach channel or other local materials within the forebay area. With the diversion channel safely established at elevation of 135 feet, move the crane down the temporary ramp to the right channel area. Remove the sheet-piles and pervious backfill used for the crane pad.

h. Using the crane on the right abutment or in the right channel area, demolish the south (right) spillway, and excavate the concrete gravity section to an elevation of 170 feet. Remove the timber access bridge, five spillway gates, and gate hoist using the crane(s) and remove the materials from the site using flatbed trucks. Drill and blast the reinforced concrete spillway structure in 1 or 2 shots, and excavate the upper portion of the concrete gravity section in 3 or 4 layers. Remove an estimated 2,200 yd³ of reinforced concrete rubble from the spillway location, and an estimated 2,200 yd³ of concrete rubble from the upper portion of the gravity section, to an approved disposal site. Also remove the remaining portions of the concrete channel lining (gunite cap) from the forebay area (400 yd³).

i. Construct a short ramp and an access road from the right channel area, across the south spillway foundation and lowered gravity section, to the left abutment. Limit the maximum grade to 15 percent. The width of the gravity section at an elevation of 170 feet is about 28 feet. Move the crane from the right channel area to a left abutment crane pad above the existing powerhouse.

j. Construct a downstream cofferdam within the river between the powerhouse and the spillway channel as needed to unwater the powerhouse tailrace area. An estimated volume of 3,500 yd³ would be required to provide an embankment having a 10-foot crest width and 2:1 slopes. Install a geomembrane lining to serve as a water barrier.

k. Remove the surge tank, remaining portions of the penstocks, and all mechanical and electrical equipment from the powerhouse using the left abutment crane and bulldozer. The larger items must be cut down to more manageable sizes for handling purposes. Place items at a suitable point for removal using the right abutment crane, or for hauling directly from the site in trucks across the lowered gravity section. Remove all window glass from the powerhouse. Demolish the powerhouse superstructure and dump an estimated 2,600 yd³ of reinforced concrete rubble onto the penstock site. Blast the powerhouse substructure concrete (5,000 yd³) into the tailrace basin. This will minimize hauling requirements and provide suitable materials for channel backfill at the site.

Stage Three — Dam and Hydraulic Fill Removal (See Drawing 1440-D-152)

a. Move the crane from the left abutment across the lowered concrete gravity section to the right channel area. Leave the bulldozer on the left abutment. Remove the temporary ramp fill (from stage two) and excavate the upstream channel (hydraulic fill) materials in the dry to an elevation of 145 feet. Fill materials will mainly consist of sands and gravels, but may include

some larger rockfill and the fir mattresses, for an estimated in-place volume of 50,000 yd³. Temporary haul roads must be established within the channel for hauling fill materials to the main access road. Also demolish the concrete block between the left and right channels, and dump an estimated 1,900 yd³ of concrete rubble onto the penstock site.

b. Demolish the remaining portion of the concrete gravity (main dam) section in lifts to an elevation of 90 feet. The excavated surface of the gravity section should remain above the level of the upstream channel fill materials for flood protection purposes, until the channel fill near the dam becomes predominantly rockfill and erosion-resistant. The feasibility design assumes rockfill will be encountered at an elevation of about 145 feet, at which point the gravity section can be removed to the final grade. A 2:1 slope is assumed in the rockfill between elevation of 145 feet and elevation of 90 feet just upstream from the dam, bounded by the existing rock cliffs on both ends. An estimated 14,500 yd³ of concrete rubble (based on an assumed swell factor of 1.35) will be removed from the dam, to be dumped onto the penstock site using the crane and a skip for later placement as backfill within the diversion channel.

c. Perform any final shaping or excavation work necessary within the river channel between the damsite and the downstream cofferdam, especially the concrete rubble at the powerhouse site, and possibly the concrete-filled caisson near the toe of the gravity section. Breach the downstream cofferdam and place the rockfill/riprap along the left side of the river channel for future erosion protection. Remove the geomembrane liner.

Stage Four — Final Channel Excavation and Site Cleanup (see Drawing 1440-D-153)

a. Following the spring runoff period (May and June), excavate the rockfill materials within the original river channel from the damsite to the diversion channel in about 2-foot increments from elevation of 145 to 135 feet, diverting streamflow from the diversion channel to the original river channel. A portion of the excavated materials may have to be stockpiled at the penstock site above the diversion channel for subsequent backfilling of the channel.

b. During the low-flow period (August through October), complete the excavation of channel fill materials along the original river channel and through the upstream landslide area as required to establish the final streambed grade. The total in-place volume of channel fill materials below elevation of 145 feet is estimated to be about 45,000 yd³ of rockfill (near the dam) and about 45,000 yd³ of sands and gravels (further upstream).

c. Demolish the intake structure on the left abutment by blasting in lifts, and push the concrete rubble into the diversion channel using the bulldozer. Shape the backfill materials within the diversion channel/north spillway channel/penstock site to the desired contours. Spread stockpiled material (dirt, rocks, etc.) across the left abutment area to a minimum depth of 2 feet over waste concrete materials, and provide desired final, natural-looking contours with slopes of 2:1 or flatter. Place topsoil over the left abutment site as required and seed. Remove the bulldozer from the left abutment area by airlift or other means as necessary.

d. Remove all remaining site buildings (maintenance shop building on right abutment, garage on left abutment), fencing, and other improvements as required.

5. Excavation of channel fill — As described previously, the channel fill consists primarily of hydraulic fill (sands and gravels) and large rockfill, but includes fir mattresses and a reinforced concrete cap. The channel fill materials may be excavated by mechanical means as discussed in the previous section, much of which to be performed either underwater or in saturated conditions. Significant cost savings may be realized if some of the channel fill materials are permitted to erode downstream, particularly within the approach channel for the diversion channel above elevation of 135 feet, rather than requiring mechanical excavation and removal to land disposal sites. This alternative will be evaluated further during final design, using geologic data additional to that currently available. An estimated potential savings of about \$800,000 could be realized if 50,000 yd³ of sand and gravel could be eroded rather than mechanically excavated and transported to a disposal site.

6. Waste disposal — Suitable sites for disposal of waste concrete and materials (dirt, rocks, etc.) are being investigated within 10 miles of Elwha Dam. On-site burial of up to 25,000 yd³ of concrete, including the retention of the north spillway gravity walls, the penstock saddles and thrust blocks, and minor concrete placements, result in an estimated potential savings to the project of about \$500,000. The cost estimate assumes all waste concrete removed from the damsite is dumped at an approved disposal site without further handling. Additional costs to crush concrete to an average 1½-inch size for off-site disposal (with any reinforcing steel removed) may range from about \$7 to \$14 per cubic yard, depending upon the size of the broken fragments. If such crushing is found to be necessary, the costs for removal of Elwha Dam could increase by an estimated \$100,000 or more.

The steel penstocks, surge tank, Tainter gates, and slide gates would probably be salvaged for recycling. Hazardous materials would be sent to an approved disposal site. Other items would be salvaged for historical or commercial value, or buried in a landfill.

7. Site restoration — All waste materials remaining on site would be buried under a minimum of 2 feet of material (dirt, rocks, etc.) and shaped to natural-looking contours. Materials suitable for use as topsoil would be spread over the site and seeded as required. Normal rainfall rates should be sufficient to support revegetation.

Following site restoration, the river channel upstream from the damsite should be able to contain streamflow up to 50,000 cfs below the diversion channel invert at elevation of 135 feet. This flow rate would correspond to greater than a 50-year flood. The seepage potential through the channel backfill on the left abutment would therefore be fairly remote, but not impossible. If necessary, a concrete cutoff wall could be constructed within the diversion channel to provide full protection from channel seepage.

Figure 11. — Elwha Dam, Diversion Channel Discharge Curve

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B. GLINES CANYON DAM

1. Decommissioning of powerplant — This would include any work at the Glines Canyon Dam powerplant required in advance of the dam removal contract work, including de-energizing the electrical equipment, removing hazardous materials, rerouting transmission lines, and removing the turbine runner from the main unit to permit no-load releases through the power penstock. Hazardous waste collection and disposal is discussed in Chapter VIII, Section A.

2. Site access and mobilization — The left (west) abutment of Glines Canyon Dam is accessed by an existing paved, two-lane road (Olympic Hot Springs Road) from U.S. Highway 101. A bridge located downstream from the dam across the Elwha River is suitable for highway loads. One lane of the existing road within the park was lost in March 1995 due to a slope failure along the river, with access limited to the remaining lane. This study assumes permanent repairs will be completed to the access road prior to award of the dam removal contract. Several site-access improvements may be required for about the last 0.2 miles of permanent road up to the left abutment. These include widening (as necessary) to accommodate construction traffic and normal maintenance of the roadway surface and drainage culverts during the contract period. Access roads to the powerhouse and the switchyard are provided from the main access road and are unpaved. Additional improvements to the main access road are discussed in Chapter VIII, Section C.

Access to the right abutment of Glines Canyon Dam is provided by an existing unpaved, single-lane road (Whiskey Bend Road) which branches off from the main access road near the Elwha Ranger Station. Contractor access to the right abutment of the dam is not currently expected to be required.

A temporary haul road may be required along the river channel between the dam and the powerhouse tailrace for removal of waste concrete, rock, and other debris during the low-flow period following dam removal.

Mobilization costs are estimated to represent about 5 percent of the cost of dam removal, plus an additional estimated \$160,000 for mobilization of a large ringer crane (which for stability and load distribution uses a circular support pad rather than outriggers) with 300-foot boom to the left abutment. A large crane pad and contractor staging area would be required on the left abutment, to be constructed using materials from the existing earth embankment at the site. The ringer crane will require a level pad approximately 48 feet in diameter (36-foot-diameter ring plus 6 feet of clearance around the periphery). Several large trees must be removed to permit full access of the site using the crane.

Construction power is assumed to be supplied by a single 150 kVA engine-generator, although alternate sources of electric power may be available. The cost of supplying electric power from the local utility (Clallam County Public Utility District) or from Bonneville Power Administration will be investigated during final design.

The current estimate assumes no constraints on construction activities will be necessary for protection of Threatened and Endangered species. Potential nesting sites for the spotted owl and marbled murrelet (both Threatened and Endangered species) exist in the vicinity of the project. Observation of these nesting areas by the Service occurred during the 1995 nesting season (May 1 to August 15) and are planned for the same period in 1996. If these nests are found to be active, “permits to take” (issued by the Service) should be investigated in lieu of placing noise limits and other constraints on the construction activities. Estimates of blasting parameters for removal of Glines Canyon Dam were

developed by Reclamation for use in modeling environmental noise levels during construction. The results of this modeling are reported in Appendix G.

The NPS may decide to close the Elwha basin to all visitor use during the demolition contract, to minimize operational problems. This would include the closure of campgrounds and restriction of vehicle use. A shuttle service could be operated to transport visitors to upper trailheads and for sightseeing. The cost estimate assumes the contractor's demolition activities would not be adversely impacted by park visitors.

3. Streamflow diversion — A series of ungated notches will be excavated within the arch dam section below the draw-down limits of the existing power penstock, to lower the reservoir level for removal of the dam. Details of the proposed diversion plan are provided in the following paragraphs. The discharge capacity of the ungated, 25-foot-wide notches is shown in Figure 12. Average daily discharges are assumed for purposes of estimating reservoir levels and diversion requirements.

The contractor is assumed to begin site mobilization in November. Glines Canyon Dam would be operated for flood control to the maximum possible extent until July of the following year, passing normal inflow through the power penstock (and spillway, if necessary) while using available reservoir storage in Lake Mills to reduce flood peaks. Reservoir releases would be controlled as necessary to facilitate construction of the surface diversion channel at Elwha Dam. Following completion of the surface diversion channel at Elwha Dam, excavation of the arch dam section at Glines Canyon Dam would commence, with the reservoir held at the lowest possible level during the demolition activities.

When the arch dam has been excavated to the reservoir level, a single notch having a bottom width of 25 feet will be excavated at the left abutment to a maximum depth of 15 feet below the reservoir level by controlled blasting. Initial blasting will be confined within the limits of the notch between the downstream face and no closer than 3 feet from the upstream face, with blasting mats used to contain the concrete rubble on the dam crest for removal. The final blasting will remove the upstream face of the notch in four segments, limiting the rate of increase in reservoir outflow to about 1,000 cfs per hour. The peak flows will attenuate quickly as the reservoir level drops. The rate of reservoir drawdown will be dependent upon the inflow and reservoir storage capacity, but should generally permit lowering the reservoir level about 8 feet within two days. Downstream flood warnings will be provided prior to blasting each notch.

Notches (for streamflow diversion) would continue to be excavated from the arch dam section at 7.5-foot increments between elevations of 522.83 and 410.33 feet, alternating between the left and right abutments. Assuming no demolition work would be performed during the three months total from November 15 to December 15 and May 1 to June 30, average inflow during the remaining 9 months of the year would be about 1,300 cfs. Although the construction schedules and cost estimates assume an average notch width of 25 feet and a minimum discharge capacity of 1,400 cfs with 7.5 feet of head, actual notches may be sized according to diversion requirements. Reservoir inflow in excess of the notch capacity would cause the reservoir level to rise and demolition work to be temporarily suspended. Overtopping of the partially-excavated arch dam section during flood flows would not cause damage to the structure or downstream channel.

The reservoir level would drop in 7.5-foot increments at intervals ranging between one week and one month, depending upon the layer elevation and reservoir inflow. Excavation of the lower portion of the arch dam section (including the final two layers and the base of the dam) would be performed

during August and September of the second year when streamflow would be at a minimum. Final channel cleanup would be completed by late October.

4. Structure removal — Each structure at Glines Canyon Dam was evaluated for possible retention, based on anticipated effects on ecosystem and fisheries restoration, historic and cultural mitigation, visitor safety, maintenance costs, demolition costs, esthetics, and opportunities for site interpretation. Structures recommended for removal from the site include the arch dam section, the penstock intake structure/trashrack tower, the surge tank tower, the transformer yard and shed, the left abutment earthfill embankment, and the boat house. Other structures will be retained, with necessary modifications, as described in Section 6 below. The structure removal plan for Glines Canyon Dam consists of four stages as described below, and as depicted on Drawings 1440-D-154 and 1440-D-155.

Removal of the arch dam section is assumed to be performed using both conventional blasting and diamond-wire sawcutting methods, for estimating purposes. Diamond-wire sawcutting will provide smooth surfaces for the concrete structures to be retained on both abutments, facilitate excavation of reinforced portions of the arch dam section, improve control of the excavation grade, provide smooth working surfaces for excavation of each layer, and permit removal of the concrete in large blocks (rather than attempting to confine rubble to the working surface and removing the rubble by loading skips).

Stage One — Diversion Through Penstock, Remove Layers 1-7 (see Drawing 1440-D-155)

a. Draw the reservoir down to its lowest possible level using the spillway and the power penstock. The existing spillway will remain available for the passage of large floods above the spillway crest, elevation of 570.33 feet. The power penstock is assumed to remain available for reservoir releases to at least elevation of 525 feet, below which some air entrainment may be seen as a vortex at the reservoir surface. Remove the log boom as required. Operate for flood control until the surface diversion channel is completed at Elwha Dam.

b. Excavate the earthfill embankment on the left abutment (having an estimated volume of 2,400 yd³) to provide materials for construction of a crane pad (requiring about 150 yd³) and contractor use area on the left abutment above elevation of 600 feet. Embankment materials in excess of those required at the site would be removed.

c. Erect a ringer crane with a 300-foot boom on the left abutment crane pad. Deliver any necessary demolition equipment and materials to the dam. While reservoir releases are limited to the power penstock, drill and blast the concrete apron at the toe of the dam for later removal. An estimated 500 yd³ of concrete rubble will result from the blasting. The rubble will provide sufficient protection for the dam foundation during future diversions over the dam crest.

d. Following completion of the surface diversion channel at Elwha Dam, begin demolition of the arch dam section in 7.5-foot-high layers (as shown in Drawing 1440-D-155) using diamond-wire sawcutting methods. Make horizontal cuts every 7.5 feet by sawcutting between the thrust block and pier 1 of the gated spillway. Make transverse (upstream-downstream) vertical cuts by sawcutting within reinforced areas of the dam, or by drilling closely-spaced blast holes for pre-splitting unreinforced concrete. Steel reinforcement will be encountered throughout layer No. 1 (elevations of 590.33 to 582.83 feet) and in portions of layers 2 through 7

(elevations of 582.83 to 537.83 feet) near the abutments. Sawcutting rates should average about 7.5 ft² per hour,² with a continuous operation assumed for each single cut. The horizontal sawcuts will determine the required duration for removal of each layer. Vertical sawcuts will establish smooth surfaces at the thrust block and spillway pier 1 to be retained.

e. Sawcut and/or pre-split the arch dam concrete from layers 1 through 7 into large blocks up to the safe crane capacity for the block location, and within the allowable load limits for hauling from the damsite in trucks. Typical blocks will weigh about 35 tons, with a volume of up to 18 yd³ each. For the 7.5-foot-high blocks, typical widths and lengths will average 8 feet; however, concrete blocks within reinforced areas of the dam will range up to 14 feet long, since no longitudinal (cross-canyon) saw cuts can be made. One longitudinal, pre-split cut may be made within unreinforced portions of layers 5 through 7. Excavation rates should result in the removal of an average of two concrete blocks (or 36 yd³) per day. The ladder well walls may be excavated with the blocks, or by themselves in advance of the dam removal.

f. Remove any concrete remaining on the bedrock abutments following block removal using a hoe-ram or other methods. Some grouted rock bolts may be required to be installed on the abutments to ensure stability of the remaining rock mass. The cost estimate includes nominal quantities for concrete demolition on abutment contacts (100 yd³) and for 1-inch-diameter grouted rock bolts (ten 20-foot-long bolts).

Stage Two — Diversion Through Notches, Remove Layers 8-12 (see Drawing 1440-D-155)

² Thus, if the section the dam to saw was 7.5 feet wide, in one hour the cut would be 1-foot deep.

a. Notches are to be excavated below Layer 7 at 7.5-foot intervals (alternating from left to right abutment) to provide 16 overflow sills between elevations of 522.83 and 410.33 feet for streamflow diversion. Flood flows exceeding the capacity of the notches will overtop the remaining portions of the excavated dam crest, requiring a temporary work shutdown. The notches will be excavated below reservoir levels in stages to depths of 15 feet, to permit as much demolition work as possible to be performed in the dry. Lines of 2-inch-diameter, pre-split holes on 1.5-foot centers were assumed for estimating purposes to be drilled along the inside end of each notch (nearest the ladder well) and about 3 feet inside the upstream face of the dam (near the abutment) to set the limits for production blasting. Production blasts (using 2-inch holes on 4-foot centers for estimating purposes) will be controlled and confined by mats or nets for removal of the resulting concrete rubble from the dam crest, rather than from the river channel below. Breaching of the notches will be performed by blasting the remaining upstream face in segments to control reservoir release rates. The resulting rubble would be washed downstream. Excavation of a typical notch having a bottom width of 25 feet may be performed in four segments at 1-hour intervals to limit the rate of increase in reservoir outflow to about 1,000 cfs per hour. (This would be similar to the maximum hourly rate of increase experienced during passage of the 5-year flood.) Each notch would be designed to lower the reservoir (within 1 or 2 days) to a level required for excavation of the next layer. A 25-foot width would have a discharge capacity of about 1,400 cfs with a 7.5-foot head. Actual notch widths may be dependent upon site conditions, projected inflows, and sediment management objectives. A new notch can be established about every 8 to 16 days, assuming no flood delays.

b. Sawcut and/or pre-split the arch dam concrete in layers 8 through 12 as notch sills 1 through 5 are completed and the reservoir level is lowered. Layer 8 will require a horizontal sawcut at elevation of 530.33 feet from the right abutment to the first notch. Layers 9 through 12 will require horizontal sawcuts between the notches on both abutments. The removal plan assumes one longitudinal pre-split for layers 8 through 10, and two longitudinal pre-splits for layers 11 and 12, with transverse pre-splits as required to produce suitable block sizes for removal by the ringer crane.

c. Upon completion of notch sill 5 to elevation of 492.83 feet, the penstock guard gate can be closed and demolition work can begin on the intake tower and the surge tank. The intake tower and trashracks are to be removed, and a concrete plug is to be constructed at the upstream tunnel portal. The surge tank is also to be removed, with a concrete plug placed around the penstock at the downstream tunnel portal, and a steel plate added to the top of the penstock at the opening for the tee connection. A concrete cap may also be required at the gate shaft.

d. Sediment releases will occur with the blasting of notches in the dam. A winter shutdown to minimize impacts to anadromous fish is assumed in the construction schedule for the 1-month period November 15 to December 15.

Stage Three — Diversion Through Notches, Remove Layers 13-21 (see Drawing 1440-D-155)

a. Sawcut and/or pre-split the arch dam concrete in layers 13 through 17 as notch sills 6 through 10 are completed and the reservoir level is lowered. Layers 13 through 17 will require horizontal sawcuts between the notches on both abutments. The removal plan assumes two longitudinal pre-splits for layers 13 through 17, with transverse pre-splits as required to produce suitable block sizes for removal by the ringer crane.

b. Reservoir sediments are expected to be encountered at the upstream face of the dam for removal of layers 18 through 23. No horizontal sawcutting is assumed below elevation of 462.83 feet. Drill holes were assumed to be located on 4-foot centers for production blasting of the concrete in layers, with blasting mats used to confine the concrete rubble to the dam crest for removal using the ringer crane and a skip. Layers 18 through 21 will be blasted as notch sills 11 through 14 are completed and the reservoir level is lowered.

c. Remove low-level sluiceway trashrack, butterfly valve, and valve operator during demolition of layers 19 and 20. The concrete encasement will be blasted, and the metalwork will be lifted out by the ringer crane for disposal.

d. A summer shutdown to minimize impacts to anadromous fish is assumed in the construction schedule for the 2-month period May 1 to June 30.

Stage Four — Remove Final Layers and Base of Dam (see Drawing 1440-D-155)

- a. Excavation of the final two layers (22 and 23), excavation of the base of the dam to an elevation of 400 feet, and final channel cleanup (including removal of any concrete rubble, boulders, and the tailrace training wall) must be performed during the low-flow period (August through October).
- b. Drill holes were assumed to be located on 4-foot centers for production blasting of the concrete in layers 22 and 23, with blasting mats used to confine the concrete rubble to the dam crest for removal using the ringer crane and a skip. Layers 22 and 23 will be blasted as notch sills 15 and 16 are completed and the reservoir level is lowered. Notch sill 16 will be considerably narrower than the other notches, due to the physical dimensions of the canyon near the base of the dam.
- c. Vertical and angled drill holes will be required from an elevation of 417.83 feet for production blasting of the base of the dam to the final excavation level (elevation of 400 feet). Angled holes are required for excavation of the sloping downstream face of the arch dam at that elevation, and to remove the concrete below notch sill 16 at elevation of 410.33 feet.
- d. Remove as much concrete rubble as practical from the river channel during the low-flow period, including the rubble from all notch excavations in addition to the rubble from the base of the dam. Also remove the tailrace training wall at the powerhouse.
- e. Final site cleanup will include removal of the transformer yard equipment, metal shed, and fencing, and shaping and seeding of the construction areas as necessary.

5. Waste disposal — All waste materials will be removed from the site. Suitable sites within 20 miles of Glines Canyon Dam for disposal of waste concrete and material (dirt, rocks, etc.) are being investigated. The cost estimate assumes waste concrete will be dumped at an approved disposal site without further handling. About 7,000 yd³ of concrete will be in large blocks, weighing between 26 and 35 tons each. Additional costs to crush all waste concrete to an average 1½-inch size for disposal (with any reinforcing steel removed) may range from about \$7 to \$14 per cubic yard or more, depending upon the initial size of the waste concrete. If such crushing is found to be necessary, the estimated costs for removal of Glines Canyon Dam could increase by more than \$200,000.

Preliminary investigations were made into the potential use of the concrete blocks and rubble for beneficial uses, including shoreline protection and off-shore reefs. Initial indications were for a predominant size of 1 meter (3.3 feet), with a maximum size up to 2 meters (6.6 feet). Further investigations of potential beneficial uses of waste concrete will be made during final design.

The steel trashracks, surge tank, and butterfly valve would probably be salvaged for recycling. Hazardous materials would be sent to an approved disposal site. Other items to be removed would be salvaged for historical or commercial value, or buried in a landfill.

6. Structure retention — Several structures are proposed to be retained at the site for their historical value and opportunities for interpretation. Retention of the following structures would not place any limitations on restoration of the ecosystem and anadromous fisheries, and could result in a

significant cost savings to the project. Each structure retained would require some modifications to minimize future maintenance and/or provide for visitor safety.

Retention of the right abutment thrust block, gravity section, and earthfill embankment (with log flume) would provide an excellent public overlook at the Glines Canyon site accessible from Whiskey Bend Road, and would eliminate the need for contractor access (and disturbance) on the right abutment of the dam. The existing guardrails would be modified by the addition of wire mesh to improve visitor safety, and some additional fencing and signs may be required around the site. An estimated savings of about \$500,000 could be realized by the retention of these features.

Retention of the gated spillway structure would provide significant historical mitigation and interpretation opportunities, while providing a public overlook on the left abutment of the dam accessible from the Olympic Hot Springs Road. Long-term maintenance costs would be minimal; however, some modifications would be necessary to provide for visitor safety, such as improved guardrails (addition of wire mesh), permanent covers for the openings in the hoist deck, and measures to prevent visitors from reaching the spillway chutes. The gated spillway could be retained at an estimated savings of about \$500,000.

The powerhouse would be retained as an excellent example of early hydropower generation technology. Exhibits could be located on the main floor to describe the various pieces of equipment and how water was used to generate electricity. The large glass windows would be replaced or protected to prevent vandalism, and any hazardous materials would be removed or confined (i.e., asbestos insulation). The roof would be repaired if necessary. A estimated savings of about \$1 million could be realized by retention of the powerhouse and equipment.

The steel penstock between the tunnel portal and the powerhouse also offers significant historical mitigation and interpretation opportunities. To be included in an interpretive trail, the wooden stairway on top of the penstock would be replaced by a low maintenance, metal stairway with guardrails, and the stairway and penstock would be painted. The penstock opening at the surge tank location would be permanently sealed with a steel cover plate, and the tunnel portal would be sealed with a concrete plug to prevent visitor access. A estimated savings of about \$200,000 could be realized by retention of the penstock.

Retention of the gatehouse, shaft, and footbridge is currently being considered for potential historical mitigation and interpretation opportunities. If retained, the glass windows and door would be removed to prevent vandalism, and the wooden stairway would be replaced with a low maintenance, metal stairway. The guardrails would be modified with wire mesh for improved public safety. All exposed electrical wiring and the wooden power pole would be removed, and openings in the gatehouse floor and at the top of the shaft would be sealed. A modest savings of about \$30,000 could be realized by retaining these features.

The two wood-frame residences and the maintenance shed on the left abutment would provide needed office and storage space at the site during the demolition work, and would be retained at least for the duration of the contract. The NPS may decide to retain these structures for ecosystem restoration researchers. A modest savings of about \$20,000 could be realized by retaining the structures.

7. Site restoration — A site restoration plan will be developed for the left abutment to provide natural-looking contours following removal of the earthfill embankment and the crane pad. The river channel between the damsite and the powerhouse contains large boulders and woody debris which will

be removed as necessary for fish passage and potential recreational use of the river, if possible. Fencing, guardrails, and warning signs are anticipated to be required around remaining structures on both abutments for public safety. Permanent electric power will be provided to the site for the structures and exterior lighting. Improvements to the Olympic Hot Springs Road are described in Chapter VIII, Section C.

Figure 12. — Glines Canyon Dam, Discharge Curves for 25-Foot-Wide Notches

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CHAPTER VIII. ADDITIONAL CONSTRUCTION REQUIREMENTS

A. HAZARDOUS WASTE COLLECTION AND DISPOSAL

Hazardous materials identified at the dam sites include asbestos siding and floor tiles in the maintenance shop building at Elwha Dam, asbestos insulation for wiring in both powerhouses, banks of lead-acid batteries in both powerhouses, small quantities of hazardous chemicals, and possible contaminated oil and soils at both sites. Asbestos may also be found in duct insulation, valve or turbine packing, piping or turbine gaskets, and generator brake shoes. Lubricating oils and grease should be removed from mechanical equipment and properly disposed of as one of the first steps of decommissioning. Polychlorinated biphenyls (PCBs) may also be present in small quantities at both sites. The Service has prepared a separate report on the proposed collection and disposal of hazardous waste, with an estimated cost to the project of \$1.2 million.

Of additional concern is the potential for lead-based paint to have been used for steel coatings at both sites, including penstocks, surge tanks, slide gates, Tainter gates, trashracks, and any painted equipment. Although not currently classified as a hazardous material by the Interior Department, all lead-based paint (if shown to exist) would have to be removed from steel items proposed for recycling. Estimated costs for removal and disposal of lead-based paint could exceed \$200,000 if such action is found to be required.

B. REMOVAL OF TRANSMISSION LINES

Existing 69 kV, three-phase transmission lines extend approximately 7 miles from Glines Canyon Dam to Elwha Dam (Line No. 3), approximately 6 miles from Elwha Dam to the Daishowa-America paper mill in Port Angeles, Washington (Line No. 1), and approximately 8 miles from Elwha Dam to the Bonneville Power Administration–City of Port Angeles substation (Line No. 2). Previous estimates included the removal of 14 miles of transmission lines and poles at a cost of \$238,000 plus contingencies (1990 dollars). Additional studies are being performed to determine the current project requirements and associated costs for this work.

C. ROAD IMPROVEMENT PLAN

Heavy construction traffic associated with hauling waste materials from both dam sites to designated disposal areas may cause damage to some of the existing roadways in the area. The Dam Removal Task Group was requested to determine the extent of road damage that may occur and appropriate methods of repair.

Primary access to Elwha Dam is along U.S. Highway 101 to State Highway 112. From Highway 112, Elwha Dam is located about one-half mile south on the Lower Elwha Dam Road. The two highways are designed and constructed to accept standard truck traffic. The Lower Dam Road consists of an improved gravel surface, and is almost entirely located on Elwha Dam project lands.

Primary access to Glines Canyon Dam is along U.S. Highway 101 from the east or west, and then south along Olympic Hot Springs Road for about 4.5 miles to the dam. U.S. 101 has been designed and constructed to accept standard truck traffic, including roadway surfaces and bridges.

The Olympic Hot Springs Road has only a bituminous surface treatment (BST), consisting of two coats of a chip material placed over liquid asphalt. The road base of this section of road consists of a thin layer of gravel. The subgrade contains a high percentage of clay. Approximately 3.5 miles of the road to the dam is within Olympic National Park and maintained by the NPS. The initial mile of this road off of U.S. 101 is maintained by Clallam County. The Altaire Bridge has been designed for "HS 20" loads and will easily accommodate construction traffic.

The dam removal contractor will be using conventional hauling equipment for removing the waste and salvage materials from the dam sites and transporting them to disposal areas. Based on the current information available on the location of potential disposal sites, the majority of travel will be along the two highways, except for 4.5 miles along the Olympic Hot Springs Road, and 0.5 miles along the Lower Elwha Dam Road. Truck traffic for removal of Glines Canyon Dam may involve up to 3 trips per day over the two-year construction period. Hauling would be performed 5 or 6 days per week during daylight hours along the Olympic Hot Springs Road. Truck traffic for removal of Elwha Dam may involve hauling as many as 8 loads per hour for two 10-hour shifts during the second year. In both cases, the loads would not exceed 80,000 pounds. Typical construction equipment used for hauling would include flat beds, low boys, and dump trucks. The dam removal contractor will be required to make appropriate repairs to the Lower Elwha Dam Road and the Olympic Hot Springs Road to allow for normal use during the two years of construction.

The probable type of equipment to be used for hauling materials, the expected weight of the individual loads, the number of loads, and the condition of the existing roads were the criteria considered for determining the final repairs that may be required following completion of construction activities for dam removal. This analysis indicated the existing state and major county roads are currently adequate to withstand the expected truck traffic. According to the Clallam County Road Department, Olympic Hot Springs Road from U.S. 101 to Glines Canyon Dam was not designed to carry a significant number of heavy truck loads. The clay subgrade becomes very soft and deformable when wet. These conditions will not hold up very well to the volume and weight of the truck traffic during the construction period, and may produce a non-uniform surface grade containing dips and ruts. While the dam removal contractor will be required to make any necessary repairs during construction to accommodate his activities and local traffic, major repairs will be needed following construction.

Two types of roadway repairs were considered. They consisted of either spot patching with resurfacing, and total replacement. The alternative for spot patching, as needed, is difficult to evaluate due to the inability to identify with any confidence the extent of potential damage and the amount of patching that may be required. It is anticipated that dips and ruts will be typical repair requirements, which could involve long sections of the road. The spot patching alternative would include resurfacing of the entire roadway with two layers of bituminous surface treatment. Total replacement includes the removal of the existing roadway surface by pulverization.

The pulverized material would be spread over the subgrade during the regrading process, and compacted in place. Two layers of bituminous surface treatment would be placed over the subgrade, similar to the spot patching alternative. This treatment matches the current roadway surface, and is considered adequate for the normal future traffic expected along this road. The existing roadway has functioned very well under similar traffic patterns.

The total replacement alternative ensures that all potential deficiencies are addressed, and is recommended. The roadway improvements would be performed during the first road construction season following completion of the dam removal activities. Replacement of the entire stretch of road from U.S. 101 to Glines Canyon Dam would take approximately one month. A cost estimate for replacing the entire roadway was prepared by Reclamation's Technical Service Center. The estimated field cost for performing the work described for this alternative is \$480,000. A copy of the construction cost estimate is included in Appendix E.

The Corps (1987) determined that portions of the Olympic Hot Springs Road must be raised for flood protection due to higher stages of the Elwha River. Approximately 1 mile of road within the park may be raised 1.5 feet, and approximately 0.7 mile of road outside the park may be raised by 1.5 to 2.5 feet. These requirements are discussed further in the 1987 Corps report on flood control measures.

The quantity and source of materials required to raise the Olympic Hot Springs Road will be investigated further during final design. Excavation materials from the removal of Elwha and Glines Canyon Dams may be considered to meet this requirement, which could exceed 10,000 yd³.

CHAPTER IX. PROJECT SCHEDULE AND ESTIMATED COSTS

A. BASIC ASSUMPTIONS FOR PERFORMING WORK

Basic assumptions were made for performing the demolition work at both dam sites, as follows.

Elwha Dam

1. The powerplant is decommissioned and all hazardous materials are removed from the site by others prior to award of the demolition contract. The turbine runners are removed from the vertical units (3 and 4) to increase the discharge capacity of the 15-foot-diameter penstock and ensure continuous release capability under all operating conditions as required for streamflow diversion.

2. The Eicher fish screen, fish rearing pens, and associated equipment and materials (including the derrick crane) are removed from the site by the owners (EPRI) prior to contract award.

[Note: In the period since these assumptions were formed, it appears the majority of the materials will be left on site. However, the additional cost to remove them is not considered to be a significant addition to the total budget, and the cost estimates in Appendix E have not been changed.]

3. All structures at the site are to be demolished. Burial of some concrete rubble and reinforcing steel will be permitted at the site to minimize project costs.

4. A suitable land disposal site is located within 10 miles of the damsite for excess waste materials (earth and concrete rubble). Waste materials are dumped at the disposal sites without further compaction or reduction in size.

5. Glines Canyon Dam will be retained to provide some flood protection until the left abutment surface diversion channel can be established.

6. Threatened and Endangered species or other environmental concerns which would affect or limit construction activities are not identified. However, to minimize impacts to anadromous fish runs, no construction activities which would introduce additional reservoir sediments to the Elwha River will be performed during the 1-month period from November 15 to December 15 and the 2-month period from May 1 to June 30.

7. Construction and operation of a slurry pipeline at the site would not have a net adverse impact on the demolition activities.

Glines Canyon Dam

1. The powerplant is decommissioned and specific hazardous materials are removed from the site as required prior to award of the demolition contract. The turbine runner is removed to increase the discharge capacity of the power penstock and ensure continuous release capability under all operating conditions as required for streamflow diversion.
2. The dam will be operated for flood control until the diversion channel is completed at Elwha Dam.
3. Only the concrete arch portion of the dam, the intake tower, surge tank, transformer yard, tailrace training wall, log boom, earthfill embankment on the left abutment, boat house, and transmission lines are required to be removed. Other structures at the site on both abutments are to be retained for interpretive purposes and historic mitigation. All waste materials are to be removed from the site.
4. A suitable land disposal site is located within 20 miles of the damsite for waste materials (earth and concrete rubble). Waste materials are dumped at the disposal sites without further compaction or reduction in size.
5. Threatened and Endangered species, or other environmental concerns, are not identified which would affect or limit construction activities, except that no construction activities which would introduce additional reservoir sediments to the Elwha River will be performed during the 1-month period from November 15 to December 15, and during the 2-month period from May 1 to June 30. Sawcutting operations at the dam are assumed to continue around-the-clock for each layer as required.
6. Construction and operation of a slurry pipeline at the site would not have a net adverse impact on the demolition activities.
7. Final improvements required by the NPS for use of the area by the public (such as road repaving, permanent powerlines, interpretation centers and displays) will be included under separate completion contracts.

B. DEVELOPMENT OF CONSTRUCTION LOGIC AND DURATIONS

Construction schedules for removal of Elwha and Glines Canyon Dams were prepared using the software Primavera Project Planner, developed by Primavera Systems, Inc., and are included in Appendix D. Construction activities and sequences were identified using the proposed dam removal plans described in Chapter VIII and the basic assumptions listed above. Activity durations were based on engineering judgement and experience, and were modified as necessary for high flows (based on average streamflow conditions and available discharge capacity). Key assumptions for establishing activity durations include an average diamond-wire sawcutting rate of 180 ft² per day for concrete in the arch at Glines Canyon Dam, and an average excavation rate of 1,000 yd³ per day for the channel fill materials at Elwha Dam. Proposed reservoir drawdown rates for Lake Mills and Lake Aldwell are also important factors affecting slope stability, sediment management, and public safety. Early and late start dates and finish dates were established for all activities based on the assumed durations and sequences. A critical path was identified for activities which could not slip without affecting the overall completion date.

Various contract start dates were assumed and the corresponding schedule impacts were evaluated on a seasonal basis to determine the optimum start date of November 1. Excavation of the diversion channel plug at Elwha Dam, and final excavation within the river channel at Elwha and Glines Canyon Dams, are scheduled for the historic low flow period (August through October) of the first and second year. Planned work stoppage periods are assumed for November 15 to December 15 and for May 1 to June 30 of each year for those activities which would release reservoir sediments to the Elwha River. These periods correspond to historic high flow periods, and are intended to minimize potential adverse impacts to anadromous fish runs (chum in winter and steelhead in summer). All work under the demolition contract should be completed within approximately two years.

C. EFFECTS ON SEDIMENT MANAGEMENT AND WATER QUALITY

Sediment routing models for the Elwha River were developed by the Sediment Management Task Group (Reclamation, March 1996a), based on the current proposed construction schedules for dam removal. The rates of reservoir drawdown resulting from the proposed dam removal plans appear to be compatible with the sediment management objectives, particularly the notching rate at Glines Canyon Dam for lowering Lake Mills. These results were used in *Report of the Water Quality Task Group* (Reclamation, in publication) to confirm the water treatment requirements. See the reports of both task groups for further details.

D. COST ESTIMATES FOR DAM REMOVAL

1. Field costs — Construction cost estimates have been prepared to a feasibility level for the removal of Elwha and Glines Canyon Dams, and related appurtenant structures as required, and are included in Appendix E. These estimates of field costs are based on computed quantities and on unit prices developed at an April 1995 price level. The cost estimates include contractor mobilization, streamflow diversion provisions, and dam and associated structure removal and disposal, with an unlisted item allowance of 5 percent and an allowance of 20 percent for contract contingencies (unforeseen costs or conditions). The methods assumed for streamflow diversion and structure demolition are detailed in Chapter VII. Cost estimates do not include costs for hazardous waste removal and disposal, powerplant decommissioning, and removal of transmission lines, which have been estimated by others. A project acquisition cost of \$29.5 million was previously established by Public Law 102-495.

The estimated field cost for removal and disposal of Elwha Dam and all appurtenant structures is \$10.5 million. The estimated field cost for removal and disposal of the arch dam, intake tower, surge tank, left abutment dike, and transformer yard at Glines Canyon Dam is \$9.7 million. The total estimated field cost for removal of both dams is therefore \$20.2 million. Assuming a two-year construction period beginning in November of the first year, expenditures for dam removal are estimated to range from 36–53 percent the first fiscal year, 47–59 percent the second, and 0–5 percent the third, based on cost-loading the early and late construction schedules discussed in Section B above.

The estimated field cost for removal of both dams is \$3.17 million (about 20 percent) higher than the total cost identified in *The Elwha Report*, and does not include costs for final design and construction management. This discrepancy is due to larger quantities and/or higher unit prices for many of the individual items of work. For example, the channel fill excavation quantity at Elwha Dam is

currently estimated at about 200,000 yd³, which is much greater than the 80,000 yd³ estimate included in *The Elwha Report* (although comparable to the 223,000 yd³ estimate included in the *Draft Staff Report*). Combined with higher unit prices for excavation and disposal, the cost of removal of the channel fill alone is \$1.2 million higher in the current estimate. The estimated cost for removal of the steel penstocks and surge tank at Elwha Dam is nearly \$900,000 higher, and the estimated cost of removal of the powerplant equipment is nearly \$500,000 higher. Unit prices for excavation and disposal of reinforced concrete structures range from about \$160 to \$210 in the current estimate, compared to \$50 to \$75 for *The Elwha Report*, increasing total estimated costs by about \$1 million. The largest single difference between the two estimates is in the cost of excavation and disposal of the arch dam section at Glines Canyon Dam. The current estimate determined costs for sawcutting, controlled blasting, and hauling of the concrete arch in layers to be about \$5.4 million (for 15,200 yd³), compared to about \$900,000 (for 11,850 yd³). Some project savings are realized by the retention of selected structures at Glines Canyon Dam (about \$2.25 million; see Chapter VII, section B.6). Costs for final design and construction management are discussed below.

2. Design costs — The total estimated cost for Reclamation's Technical Service Center to prepare final designs and specifications is \$710,500, based on an estimate of 1,155 staff-days and on estimated non-labor costs of \$17,500. This represents an engineering design cost of 3.4 percent of the contract cost. Additional costs will be incurred by Reclamation's Pacific Northwest Regional Office for the collection and preparation of design data.

3. Construction management costs — Construction management costs for maintenance of a field office for inspection services, and for contract administration and design support during the demolition work, are estimated to represent 15 percent of the contract cost, about \$3,150,000. This includes an estimate of 496 staff-days and a total cost of \$312,600 for Reclamation's Technical Service Center for design support, 1.5 percent of the contract cost.

CHAPTER X. CONCLUSIONS

Feasibility-level designs and estimates have been prepared for the removal and/or burial of all structures at Elwha Dam, and for the removal of the arch dam section, intake structure, surge tank, left abutment dike, boathouse, and transformer yard at Glines Canyon Dam. The current studies confirm that dam removal is technically feasible at both sites, and can be safely performed in a manner compatible with sediment management requirements and project objectives. Dam removal activities will require a period of approximately two years to complete, for an estimated total field cost of \$20.2 million, including an allowance of 20 percent for contingencies. Design and construction management costs for dam removal would add about 20 percent, or approximately \$4 million to the total estimated cost.

Under the proposed dam removal plans, both powerplants would be decommissioned prior to award of the demolition contract, and the turbine runners would be removed from the vertical units at both sites for improved streamflow diversion capacity. Glines Canyon Dam would be operated for flood control until a surface diversion channel is established at Elwha Dam, at which time demolition of both dams would commence.

The removal of Elwha Dam requires the demolition of about 30,000 yd³ of concrete in the gravity section and appurtenant structures, and the disposal of about 2,500,000 lbs of mechanical and electrical items in the spillways, powerplant, penstocks, and surge tank. An estimated 200,000 yd³ of sand, gravel, and rockfill must be removed from the channel forebay area to restore the river to its original course. Concrete demolition and diversion channel excavation will be performed by drill-and-blast methods, with much of the concrete and some rockfill remaining on site to provide the final site contours. Waste concrete and channel fill materials in excess of the quantities required at the site will be hauled to a disposal site within 10 miles of the damsite. Mechanical and electrical items will be salvaged for recycling, or disposed of in a landfill. Additional estimated costs for disposal of materials would be incurred if concrete crushing is required (over \$100,000) and if lead-based paint must be removed (over \$200,000).

The removal of Glines Canyon Dam requires the demolition of about 16,000 yd³ of concrete in the arch dam and intake structure, and the disposal of about 200,000 lbs of mechanical and electrical items in the sluiceway, transformer yard, and surge tank. Concrete demolition will be performed by a combination of diamond-wire sawcutting and drill-and-blast methods, with all waste materials removed from the site. Waste concrete and earthfill materials will be hauled to a disposal site within 20 miles of the damsite. Mechanical and electrical items will be salvaged for recycling, or disposed of in a landfill. Additional estimated costs for disposal of materials would be incurred if concrete crushing is required (over \$200,000).

CHAPTER XI. RECOMMENDATIONS

A. CONTRACTING BY NEGOTIATION

Negotiated procurement is recommended for future contracting of the demolition work, by which proposals would be solicited for selection based on both cost and technical merit. Negotiated procurement is a more flexible approach to contracting than the sealed bidding process, and is used when alternative, innovative approaches to construction are desired. This process allows confidential discussions (negotiations) with offerors within a competitive range of acceptable proposals, prior to best and final offers. Final selection of a proposal is based both on cost and on technical factors (including proposed construction methods, corporate experience, personnel qualifications, and project compliance) using established evaluation criteria. This contracting process has been successfully used by Reclamation for many projects, and should result in the optimum dam removal plan for the lowest overall cost.

B. ADDITIONAL INVESTIGATIONS FOR FINAL DESIGNS

Additional information will be required prior to initiating final designs. This information will be used to refine and evaluate some of the feasibility design assumptions, and to develop designs for features that were not included in the feasibility design. The following items are required to address specific issues at each dam.

Elwha Dam

1. The proposed access to the left abutment of Elwha Dam requires the use of barges or float bridges. An existing trail that passes through public and private lands could be developed into an access road, which would greatly simplify many activities. The feasibility and costs for developing this access should be investigated.
2. The characteristics and volume of the fill material in the forebay area have largely been developed from previous reports. Due to the large volume of this material, and the significant portion of the overall dam removal costs for removing this material, it is important to have a better understanding of the composition of the material to avoid costly surprises during the demolition process. A suitable method of sampling this material at depth needs to be developed for at least two locations within the forebay area.
3. A significant savings could be achieved by allowing up to 50,000 yd³ of sands and gravel to be eroded naturally by the river during the final stages of excavating the upstream fill materials. An evaluation needs to be made regarding the ability of the river to transport the materials during specified time periods. This volume is small compared to the sediment volumes proposed to be eroded from the upstream reservoirs.

4. The potential exists for seepage to occur through the diversion channel fill material above an elevation of 135 feet for extremely high flows following construction activities. The suitability of fill material along with concrete rubble should be investigated to determine if seepage could be allowed with proper gradation (without experiencing a piping failure), or whether a concrete cutoff wall should be provided within the fill.

5. Some information has already been obtained regarding the bedrock contour along the proposed diversion channel alignment. Since the proposed alignment is dependent on the bedrock elevations, it is important to substantiate the estimated bedrock contours. Seismic-refraction surveys and additional drill holes are proposed to develop this information.

Glines Canyon Dam

1. Geologic mapping of the canyon walls immediately downstream of the dam should be performed. This information would provide a better assessment for the need for rock bolts to ensure long-term stability following dam removal.

2. The retention of several features at Glines Canyon Dam will require more detailed designs for modifications that will result in minimal maintenance and vandalism, and provide for public safety.

General

Several additional activities need to be performed that are common to both dams.

1. Concrete coring and non-destructive testing needs to be performed to determine the relative strength of the concrete. This information is important for perspective contractors for evaluating various demolition alternatives.

2. While much information has already been obtained, additional data and drawings of the existing structures for both dams will be investigated during final design. Revised area and capacity curves will be developed using the latest survey data.

3. Studies have been performed by representatives of the Tribe for locating potential disposal sites that may be available for accepting the various kinds and volume of materials coming from the demolition activities. A list of disposal sites will need to be confirmed during final designs. Potential use of excavated materials to raise the Olympic Hot Springs Road, and recycling opportunities, should also be investigated.

4. The feasibility designs assumed that construction power would be provided by engine generators. The feasibility of using conventional power should be investigated, including an economic comparison of the two alternatives.

5. The impacts of construction activities on Threatened and Endangered species need to be reevaluated during final designs.

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